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SEISMIC CRITERIA FOR CALIFORNIA MARINE OIL TERMINALS

VOLUME 1

by
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13. ABSTRACT *(Maximum 200 words)*
The Navy and the California State Lands Commission entered into a Cooperative Research and Development Agreement for the development of seismic design criteria for waterfront construction. Both organizations face similar problems in the safe design of facilities and the need for a design guide. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicates that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected.

This document develops and expands on work that was begun by the Naval Facilities Engineering Service Center to provide seismic design criteria for waterfront construction. This report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals and seven chapters and three appendices of technical supporting material. The development of the criteria recognized the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State, and the economics of operating a commercial facility in a competitive structure. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage.

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Preface

This document presents guidance on the seismic design of marine oil terminals. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected. The goals are to:

- Ensure safe and pollution-free transfer of petroleum products between the ship and land based facilities.
- Ensure the best achievable protection of the public health, safety and the environment
- Maximize the utilization of limited resources

This document develops and expands on work that was begun by the US Navy to provide seismic design criteria for waterfront construction. It presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals. As such it recognizes the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State and the economics of operating a commercial facility in a competitive environment. Readers must recognize that this standard can not guarantee that if implemented and followed that all damaging effects will be precluded. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The establishment of design levels is more of a management decision that an engineering one. Considering the size of the State of California and the health and economic needs of its inhabitants, this guide is thought to be set at an optimal balance. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage. It consists of a criteria section, supporting technical commentary and three appendices.

INTRODUCTION

The California State Lands Commission (SLC) was created by the California Legislature in 1938 as an independent body, composed of three members-the Lieutenant Governor and State Controller, both statewide elected officials, and the Director of Finance, an appointee of the Governor. The SLC was given the authority and responsibility to manage and protect the important natural and cultural resources on public lands within the state and the public's rights to access to such lands. In managing the state's lands, the SLC provides two functions: (1) generating revenue for the state, and (2) protecting, preserving and restoring the natural values of state lands. The resources managed by the SLC are diverse and range from commercially valuable minerals such as oil, natural gas, hard rock minerals, sand, gravel, and geothermal steam to unique natural resources such as forests, grazing lands, wetlands, riparian vegetation, and fish and wildlife habitat.

The SLC's Marine Facilities Inspection and Management Division was created in response to the passage of the Lempert-Keen-Seastrand Oil Spill Prevention and Response Act of 1990 which mandated the best achievable protection of California's marine environment and created a $500 million Oil Spill Contingency Fund to help finance emergency response efforts and provide disaster relief in the event of a major oil spill. This strong mandate reflected the Legislature's recognition of the public outcry for stronger environmental protection following the tragedies caused by the MIT Exxon Valdez grounding in Alaska and the MIT American Trader oil spill off Huntington Beach. The SLC is focussed on protecting the marine environment through the prevention of oil spills because no matter how quickly the response is to an oil spill, severe and often irreparable damage occurs to the marine environment. Prevention is the least expensive form of environmental protection. Comprehensive Marine Terminal Regulations were formulated by the SLC and implemented in late 1992. The implementation of these regulations and the SLC inspection activities were responsible in reducing regulatory deficiencies. This document presents guidance on the seismic design of marine oil terminals. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected. The goals are to:

- Ensure safe and pollution-free transfer of petroleum products between the ship and land based facilities.
- Ensure the best achievable protection of the public health, safety and the environment
- Maximize the utilization of limited resources

A typical Marine Oil Terminal (MOT) includes some or all of the following components:

Pier
Wharf and dike
Bulkheads, quay walls, sheet piling
Pipeline (to the first valve inside an EPA containment area)
Pipeline supports
Bumper fendering, camels, batter piles
Mooring components including breasting and mooring dolphins and onshore dead-men
Local on pier in terminal storage tanks (but not storage tanks in backland ashore)
Hose Fuel Transfer equipment and structures
Vapor control systems
Fire suppression and detection systems
Building and other structures on the pier or wharf
Ancillary components
Riprap

Safe, effective seismic design consists of three elements - establishment of performance goals, specification of the earthquake intensity, and definition of the acceptable structural response limits corresponding to the performance goals. Although seismic load-performance requirements exist for structures and bridges, no requirements have been developed for the waterfront structures that are common at ports and marine oil terminals. There is also a lack of geotechnical guidelines for the seismic evaluation and design of waterfront structures. For example, very few standards exist for defining acceptable factors of safety against liquefaction in soil, a major cause of damage at the waterfront. While structural and geotechnical analysis tools for evaluating the occurrence of liquefaction and the response of structures currently exists, guidance standards which define what constitutes acceptable behavior under a prescribed load level have not been established.

It is important to understand that a complete design standard is composed of three major parts:

1. Development of a set of performance goals defining levels of operation required after earthquake ground motions of varying intensity and duration.

2. Specification of a set of earthquake intensities corresponding to prescribed risk levels.

3. Determination of the structural response limits at the specified seismic intensities which will ensure that damage is limited to meet the expected performance levels.

Thus, full design criteria includes definition of :

1. Performance objectives,
2. Specification of ground motion,
3. Specification of analysis procedures,
4. Evaluation of all possible failure modes of the global structure, including the soil foundation,
5. Definition of component damage mechanisms for all elements of the structure,
6. Development of allowable response limits such as strains, ductilities, and drifts to control element damage and structure performance,
7. Evaluation of economics of design,
8. Understanding of the reliability associated with definition of seismic intensity and structural performance.

In general, a reliability analysis evaluates the loading conditions with their measure of uncertainty, and the composition of the structure in terms of material properties, structural member sections used, the uncertainties in materials and construction etc. From the quantification of uncertainty one can calculate the distribution of possible performance outcomes. At the current state of knowledge, full explicit reliability analysis is an unrealistic goal. Generally implicit consideration of reliability aspects is made by coupling a high estimate of expected ground motion with a conservative estimate of structural limit states to ensure that the probability of exceedance of the limit state under the design intensity is sufficiently low.

This document develops and expands on work that was begun by the US Navy to provide seismic design criteria for waterfront construction. This report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals. As such it recognizes the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State and the economics of operating a commercial facility in a competitive structure. Readers must recognize that this standard can not guarantee that if implemented and followed that all damaging effects will be precluded. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The establishment of design levels is more of a management decision that an engineering one. Considering the size of the State of California and the health and economic needs of its inhabitants, this guide is thought to be set at an optimal balance. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage.

DEFINITIONS AND GENERAL CRITERIA

Construction Categories

Ordinary- General normal construction where operational, special enhanced life safety provisions, and spill containment factors are not involved.

Waterfront Transfer Structures- Piers and wharves directly involved in hazardous material transfer.

Essential- Facilities and component elements directly controlling operations that are required for safe operation and plant shutdown. Such facilities must operate during and after an earthquake to the extent required to control operation.

Hazardous Material Containment- Facilities and components serving to prevent the uncontrolled release of hazardous materials. These systems may be composed of a single system or a duel system with secondary containment.

Design Earthquake Levels

This report will utilize the following earthquake levels as defined events.

Level 1- An earthquake with a 50 percent probability of exceedance in 50 years exposure. This event has a return time of 72 years and is considered a **moderate** event likely to occur one or more times during the life of the facility. Such an event is considered a strength event.

Level 2- An earthquake with a 10 percent probability of exceedance in 50 years exposure. This event has a return time of 475 years and is considered a **major** event. Such an event is considered a strength and ductility event.

Level 3- An earthquake with a 5 percent probability of exceedance in 50 years exposure. This event has a return time of 949 years and is considered a **rare** event. Such an event is considered a strength and ductility event.

Level 4- An earthquake with a 3 percent probability of exceedance in 50 years exposure. This event has a return time of 1641 years and is considered a **very rare** event. Such an event is considered a containment event. **Note where ground motions from a 1641-year event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows actual return times and nominal return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	50	72	100
10	50	475	500
5	50	975	1000
3	50	1641	1700

Spill Size

Minor Spill- A spill of less than 1200 barrels of petroleum products or comparable hazardous material.

Major Spill- A spill of 1200 barrels or more of petroleum products or hazardous material.

General Performance Goals

Marine oil terminal facilities designed under this criteria are expected to perform in the following manner:

- To resist earthquakes of moderate size which can be expected to occur one or more times during the life of the structure without structural damage of significance. The facility is not expected to sustain a major interruption in operations.
- To resist major earthquakes which are considered as infrequent events maintaining life safety, precluding total collapse but allowing a measure of controlled inelastic behavior which will require repair.
- To have essential facility components required for safe operation, shutdown, and emergency operations function during and after rare earthquakes
- To preclude major spills of hazardous and polluting materials during and after very rare earthquakes.
- To utilize economic/risk analysis as an aid in decision making including determining the condition of the facility and it's remaining useful life.

- To consider the facility as a system and include the effect of all hazards on the operation of the whole facility and all subcomponents including lifelines.

Inherent in the general performance criteria are the three issues of structural response and integrity, spill containment, and functionality of essential emergency components. The first issue of structural response and integrity refers to the requirement for key elements such as piers and wharves that these structure should not only not collapse but that they must be able to perform to a deformation response limit so as to be in a condition which is repairable. The issue of containment refers to the need to preclude large spills. This can be accomplished by providing segmentation valves and secondary containment devices to limit the maximum size of the spill and containment or by strengthening primary components. The last issue of functionality controls the design of emergency components which are needed for post-earthquake control of the facility.

It should be noted that conformance to this criteria does not guarantee that significant damage will not occur. It does provide a prudent allocation of resources using the best available knowledge at the time it was written. A criterion must have sufficient prescription to serve as a minimum requirement and yet sufficient flexibility to allow for project specific considerations on issues such as the remaining useful life of an existing structure and the allocation of resources in achieving mandated requirements.

It is important that all interested parties including the State, the facility operator and concerned citizens establish a consensus in selecting design levels. The operator must recognize that safe design is in his long term interest by insuring minimization of damage and downtime. The State must recognize its requirement in providing clear minimum acceptable standards which are achievable. Concerned citizens must recognize that resources are sometimes limited and that transfer of oil is vital to the day-to-day life of the State and its economic viability. This document is presented as the first step in achieving that balance.

GROUND MOTION CRITERIA

A probabilistic site seismicity study is required for determining the ground motion associated with analysis of marine oil terminals. The objective of a seismicity study is to quantify the characteristics of ground shaking and the recurrence of potentially damaging ground motions that pose a risk at the site of interest. The approach taken in engineering practice is to use the historical epicenter data base in conjunction with available geologic data to form a best estimate of the probability distribution of site ground motion. Acceptable procedures for conducting a site seismicity study must include the following elements. The process consists of building a model of the region to capture the seismic activity using probabilistic procedures. The procedure consists of:

- Evaluating the regional tectonics and geologic settings
- Determining and defining seismic sources in the region of interest

- Estimating the seismic slip rate along faults in the region
- Defining the study boundaries beyond which earthquakes pose no significant damage potential to the site
- Developing an epicenter data base of historical earthquakes in the region of interest
- Specifying and formulating the site seismicity/faulting model
- Developing the earthquake (regional and fault specific) recurrence models
- Determining the maximum credible earthquakes for specific source
- Selecting appropriate ground motion attenuation relationship
- Computing the contribution of individual faults or source zones to ground motion estimates
- Combining the source contributions for all faults
- Developing probability distribution for firm site
- Determining local site soil conditions
- Determining the local site response
- Developing site specific time histories or response spectra for causative events

The supporting technical material found in Chapter 1 will present a summary and discussion of the technology for each of the elements of the analysis. Recognition of previous research in establishing recurrence parameters shall be used where available. Such bodies of knowledge are available for California from the California Division of Mines and Geology (CDMG) Internet site. Geologic slip rate data is available for a number of western faults.

Local Site Amplification As a minimum, a one-dimensional equivalent linear or fully nonlinear dynamic soil analysis shall be used to evaluate local site amplification and to determine the modification of the rock spectrum by local soil deposits. A shear-beam model representing the ground conditions from bedrock to surface is typically used, with input of the acceleration time history corresponding to the bottom boundary of the model. When the bedrock boundary slopes steeply in the vicinity of the site, such one-dimensional techniques may be inadequate

Study Results The results of a seismicity study shall include the probability of site ground motion adjusted for local site effects. The results should include a set of earthquakes including magnitude, location, and site acceleration to serve as a set of scenario events in evaluation of damage potential The structural design engineer may use either response spectra or time history techniques in the analysis of a structure.

STRUCTURAL CRITERIA FOR PIERS AND WHARVES

Performance Goals

The criteria are intended to produce a level of design in piers and wharves such that there is a high probability the structures will perform at satisfactory levels throughout their design life.

- To resist earthquakes of moderate size which can be expected to occur one or more times during the life of the structure without structural damage of significance.
- To resist major earthquakes which are considered as infrequent rare events maintaining life safety, precluding total collapse but allowing a measure of controlled inelastic behavior which will require repair.
- To preclude major spills of hazardous and polluting materials for very rare earthquakes.
- To utilize economic/risk analysis to consider alternative design
- To consider geologic hazards (e.g., liquefaction, slope stability, excessive ground settlement) as a major waterfront problem. The designer shall consider potential ground failures in the design of the structures and account for geotechnical earthquake engineering issues (change in lateral earth pressures, potential lateral movements and increased settlements).

Design Earthquakes

The pier or wharf structure shall be designed to resist the loading produced by:

- A Level 1 earthquake
- A Level 2 earthquake

In addition containment to preclude a major spill shall be provided for:

- A Level 4 earthquake

All crane rails shall be supported on piles including the seaward and the landward rail. The crane rails shall be connected horizontally by a continuous deck, beam or other means to control the gage of the rails and prevent spreading. The rails shall be grounded. For corrosion protection, it is advantageous to insulate the reinforcing steel in the piles from that in the deck.

Piers and wharves containing fueling systems shall be evaluated for a Level 4 earthquake to insure that a major spill of hazardous material is precluded. This may be accomplished by providing secondary containment systems or shutoff valves should there be breaks in fuel lines or primary containment system elements or by strengthening these elements.

- Preclude release of hazardous and polluting materials causing a major spill for a Level 4 event

Design Performance Limit States

Serviceability Limit State All structures and their foundations shall be capable of resisting the Level 1 earthquake without sustaining damage requiring post-earthquake remedial action.

Damage Control Limit State The following shall apply.

- Except as required by the following clause, structures and their foundations shall be capable of resisting a Level 2 earthquake, without collapse with repairable damage, while maintaining life safety. Repairable damage to structure and/or foundation, and limited permanent deformation are expected under this level of earthquake.

- Wharves and Piers on which hazardous materials are located shall be capable of resisting a Level 4 earthquake without a release of a major spill of hazardous materials.

Earthquake Load Combinations

Combination of Seismic Actions with other Load Cases Wharves and Piers shall be checked for the following seismic load combinations, applicable to both Level 1 and Level 2 earthquakes:

$$(1 + k)(D + rL) + E \qquad (1)$$

$$(1 - k) D + E \qquad (2)$$

where D = Dead Load

L = Design Live Load

r = Live Load reduction factor (depends on expected L present in actual case typically 0.2 but could be higher)

E= Level 1 or Level 2 earthquake, as appropriate.

k= 0.5 * (PGA), where PGA is the effective peak horizontal ground acceleration.

Note: seismic mass for E shall include an allowance for rL, but need not include an allowance for the mass of flexible crane structures.

Combination of Orthogonal Seismic Excitations Effects of simultaneous seismic excitation in orthogonal horizontal directions shall be considered in design and assessment of wharves and piers. For this purpose it will be sufficient to consider two characteristic cases:

$$100\%\ Ex + 30\%\ Ey \qquad (3)$$

$$30\%\ Ex + 100\%\ Ey \qquad (4)$$

where Ex and Ey are the earthquake (E) actions in the principal directions x and y respectively.

Where inelastic time history analyses in accordance with the requirements of Method D below are carried out, the above loading combination may be replaced by analyses under the simultaneous action of x and y direction components of ground motion. Such motions should recognize the direction-dependency of fault-normal and fault-parallel motions with respect to the structure principal axes, where appropriate.

Additional Load Combinations

Piers and wharves shall be proportioned to safely resist load combinations as shown in the following table. Each component of the structure should be analyzed for all applicable combinations. The table lists load factors to be used for each combination; the algebraic signs (+ or -) shall be those that produce the most unfavorable yet realistic loading.

$$U_i = f_D(D) + f_L(L) + f_B(B) + f_{Be}(Be) + f_C(C) + f_{Cs}(C) + f_E(E) + f_{Eq}(Eq) +$$

$$f_W(W) + f_{Ws}(Ws) + f_{RST}(R+S+T) + f_{Ice}(Ice)$$

Load Factor Design									
	U1	U2	U3	U4	U5	U6	U7	U8	U9
D Dead [1]	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
L Live [4]	1.7 [3]	1.7	1.3	1.3		1.25	[2]	1.3	
B Buoyancy	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
Be Berthing		1.7							
C Current on Structure			1.3	1.3	1.25	1.25			1.2
C_s Current on Ship			1.3	1.3	1.25	1.25			1.2
E Earth Pressure	1.3	1.3	1.3	1.3	1.25	1.25	1.0	1.3	1.2
Eq Earthquake							1.0		
W Wind on structure			0.3		1.25	0.3			1.2
Ws Wind on ship			0.3		1.25	0.3			
R + S + T				1.3	1.25	1.25			
Ice								1.3	1.2

R + S + T = Creep/Rib Shortening + Shrinkage + Temperature

Notes
1 A factor of 0.9 for checking members for minimal axial load and maximum moment
2 Depends on earthquake load
3 A factor of 1.3 for maximum outrigger float load from a truck crane
4 Concentrated live load

Vertical Accelerations

Except where preliminary analyses indicate special sensitivity to vertical acceleration effects such as in the case of use of batter piles, vertical accelerations need not be considered in design beyond the extent implied by use of Equations 1 and 2.

Methods Of Analysis For Seismic Response

Methods adopted for determining design forces and displacements shall be appropriate for the structural complexity of the wharf or pier under consideration, and shall include consideration of

- Soil/structure interaction effects,
- Natural periods of vibration of the structure,
- Effects of cracking at the elastic limit state,
- Reductions of stiffness resulting from inelastic action, where appropriate,
- Torsional response,
- Movement joints,
- Gross soil deformations,
- Liquefaction effects.

The primary purpose of the **analyses** will be to determine the maximum displacements expected under the design level earthquake. The primary purpose of **design** is to ensure that these displacements are compatible with the design performance limit state.

Method A: Equivalent Single Mode Analysis Where wharf structures are founded on essentially uniform foundation materials along the length of the wharf, where the ratio of wharf length to wharf width exceeds 3 and where the wharf deck may be considered to act as a rigid diaphragm, a simplified analysis involving amplification of the results from a single transverse modal response may be considered adequate for design and assessment purposes.

The design displacement for Method A is given by:

$$\Delta_D = k_a \Delta_T \qquad (5)$$

where

$$k_a = \sqrt{1 + (0.3(1 + {}^{20e}\!/_{L_L}))^2} \qquad (6)$$

is an amplification factor incorporating the influence of orthogonal and torsional response effects, e is the eccentricity between the center of mass and the center of stiffness in the transverse direction, L_L is the length of the wharf segment, Δ_D is the design displacement, and Δ_T is the transverse displacement corresponding to the single mode analysis.

Method B: Multi-Mode Analysis For all structures, design displacements of the wharf or pier deck may be found by a multi-mode elastic analysis. Sufficient modes shall be considered in the analysis to capture at least 95% of the participating seismic mass in both orthogonal directions. Where multiple wharf segments of similar structure and foundation conditions are linked by shear keys, it will be conservative to consider the segments as independent "stand alone" elements, except for the estimation of shear key force levels.

Method C: Pushover Analysis For all structures, 2-D nonlinear pushover analyses shall be carried out on critical frames of wharves and piers to enable the sequence of plastic hinge formation to be determined. These pushover analyses shall be used in conjunction with the design displacements determined from Method A or Method B to establish the level of inelastic rotation developed in plastic hinges under Level 1 or Level 2 earthquakes.

Method D: Inelastic Time-History Analysis As an alternative to Methods A to C, inelastic time-history analyses may be used to determine both design displacements and

inelastic rotations in plastic hinges under Level 1 and Level 2 earthquakes. A minimum of 5 spectrum compatible record sets consisting of orthogonal acceleration records shall be considered, with the mean values from the 5 analyses taken as the design or assessment levels. Each set shall have amplitude, duration and frequency appropriate for the magnitude and separation distance of the earthquake event under consideration.

Method E: Gross Foundation Deformation Analysis If geotechnical investigations indicate the possibility of gross permanent deformations of foundation material as a result of sliding on clay layers, liquefaction, or other causes, the wharf or pier shall be analyzed under the permanent foundation deformation to determine the structural displacements and internal strains and forces at critical locations.

Structural Response of Piers And Wharves

Piers and Wharves shall be designed for dependable inelastic action in accordance with the following principles:

- Inelastic response of the structure shall be limited to formation of plastic rotation in carefully detailed plastic hinges in piles.
- Shear failure of piles and inelastic action of deck members shall be proscribed by the implementation of capacity design principles, ensuring that the dependable strength of these members exceeds the maximum feasible input corresponding to the design flexural plastic hinging.
- Joints between piles and deck members shall be designed to remain essentially elastic, with recognition of the high shear forces developed within the joint region.
- Batter piles shall not be used in new design unless
  (a) the piles are designed to remain elastic under Level 2 earthquake excitation, or
  (b) a special study is undertaken to ensure that the structure, including the batter piles will respond within the specified performance limit state.

Note: The use of batter piles in wharves is strongly discouraged.

Structural Performance Limit State Strains

(a) Serviceability Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 1 earthquake shall not exceed:

Concrete extreme fiber compression strain: 0.004

Reinforcing steel tension strain: 0.010

Prestressing strand incremental strain 0.005

Structural Steel (pile and concrete filled pipe) 0.008

Hollow steel pipe pile 0.008

(b) Damage Control Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 2 earthquake shall not exceed:

Concrete extreme fiber compression strain:

Pile/deck hinge: Value given by equation 7, but <0.025

In-ground hinge: Value given by equation 7, but <0.008

Reinforcing steel tension strain: 0.05

Prestressing strand :

Pile/deck hinge: 0.04

In-ground hinge: 0.015

The design ultimate compression strain of confined concrete may be taken as

$$\varepsilon_{cu} = 0.004 + (1.4\,\rho_s\, f_{yh}\, \varepsilon_{sm})/ f'_{cc} \geq 0.005 \qquad (7)$$

where

- ρ_s effective volume ratio of confining steel
- f_{yh} yield stress of confining steel
- ε_{sm} Strain at peak stress of confining reinforcement, 0.15 for grade 40 and 0.12 for grade 60
- f'_{cc} Confined strength of concrete approximated by $1.5\, f_c'$

Structural Steel (Pile and Concrete filled pipe) 0.035

Hollow steel pipe pile 0.025

EXISTING CONSTRUCTION

The discussion of existing structures is of major importance since there are many existing terminals in use and relatively few new facilities being constructed. Many of the existing structures were built during periods when seismic standards were not well defined. In general, existing-structure performance criteria may be related to new-structure performance criteria in terms of the degree of hazard, the amount of strength required, the extent of ductility demand allowed, or the level of design ground motion. The structure once built does not "know" that it is expected to perform to a "new" or an "existing" structure criteria; it responds according the principles of structural dynamics. This guide is motivated by preservation of the environment and as such there is a mandate to use the best possible technology to ensure safe transfer of petroleum products ashore. The approach taken herein is to recognize that that the goals for both new and existing facilities should be the same. The structural parameters which are used to control the response should be the same. What is of significance is that existing structures have been in place for a period of time and have a shorter remaining life than new facilities. Thus, existing facilities face a shorter exposure to natural hazards. This major factor suggests that the design ground motions be allowed to differ based on the differing remaining life-spans of the structures. A prudent course must be charted to select reasonable goals for existing structures to minimize the adverse impact on the economics of port operations while ensuring public mandates for preservation of the environment.

The approach taken in this criteria is to utilize a factor, α, to relate the existing-structure exposure time to the new-construction exposure time taken as 50 years. The value of α is equal to or less than 1.0. The value of α is used to define the exposure time for use in the Level 1 through Level 4 earthquake return times as shown in the following sections. Determination of α establishes the seismic loading. The performance goals and response limits for existing construction remain the same as for new construction; only the loading is reduced. This applies to all elements including the structure, the foundation and all associated lifelines.

Method 1

Seismic reviews of existing waterfront construction directed by requirements of the Marine Oil Terminal Division shall utilize the above criteria for new construction as the target goal requirement for performance. In general the existing structure must satisfy the new structure performance limit states under earthquake peak ground acceleration levels corresponding to reduced exposure times as follows:

Existing -Structure Exposure Time = α (New-Construction Exposure Time)

where New-Construction Exposure Time is 50 years.

The requirement for evaluation of the seismic resistance and possible upgrade is triggered when the loading on the structure changes such as when the operation of the structure is

changed or when the structure requires major repairs or modifications to meet operational needs or when recertification is required.

Determining α When it is shown to be impossible or uneconomical to achieve new construction levels of performance, (α = 1), an economic/risk analysis using procedures shown in Chapter 6 of the supporting commentary shall be performed to determine the most appropriate facility exposure time and level of seismic design upgrade. Various alternative upgrade levels shall be considered ranging from the existing condition to the maximum achievable. Each alternative shall be examined to determine the cost of the upgrade, the cost of expected earthquake damage over the life of the structure and the impact of the damage on life safety, operational requirements, and damage to the environment. The choice of upgrade level shall be made by the design team based on a strategy consistent with requirements of life safety, operational needs, cost effectiveness, and protection of the environment. In no case shall α be less than 0.6 of the 50-year exposure time for new construction (30 years).

As a minimum analysis shall be conducted and data developed for the cases of α equal to 1.0 and 0.6. Additional cases are encouraged

Level 1- An earthquake with a 50 percent probability of exceedance in α x (50 years) exposure.

Level 2- An earthquake with a 10 percent probability of exceedance in α x (50 years) exposure.

Level 3- An earthquake with a 5 percent probability of exceedance in α x (50 years) exposure.

Level 4- An earthquake with a 3 percent probability of exceedance in α x (50 years) exposure. . **Note where ground motions from this event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows the shortest allowable return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	30	43	60
10	30	285	300
5	30	585	600
3	30	985	1000

Peer review: Review of the results of the analysis by an independent peer review panel of experts in structural engineering, geoscience, earthquake engineering, seismic risk analysis,

economics, and environmental engineering/science is required. In addition, the findings from the analysis should be reviewed by the public and other stakeholders.

Method 2

Based on the recognition that an existing structure may have a fixed life and that the upgrade is intended as limited-term solution, a reduced exposure life of 25 years may be used provided facility owners by binding agreement will take the facility out of service on or before the expiration of the 25-year period. Out of service means that the structure will not be used as a marine oil terminal for transfer of hazardous materials and that all hazardous material capable of causing a spill be removed. The 25-year period begins when CSLC approves the agreement. The following levels are to be used with Method 2

Level 1- An earthquake with a 50 percent probability of exceedance in 25 years exposure. This event has a return time of 36 years and is considered a moderate event likely to occur one or more times during the life of the facility. Such an event is considered a strength event.

Level 2- An earthquake with a 10 percent probability of exceedance in 25 years exposure. This event has a return time of 237 years and is considered a major event. Such an event is considered a strength and ductility event.

Level 3- An earthquake with a 5 percent probability of exceedance in 25 years exposure. This event has a return time of 487 years and is considered a rare event. Such an event is considered a strength and ductility event.

Level 4- An earthquake with a 3 percent probability of exceedance in 25 years exposure. This event has a return time of 820 years and is considered a very rare event. Such an event is considered a containment event. . **Note where ground motions from this event are excessive and design for major spill prevention can not be accomplished, lower levels of ground motion may used with the approval of the California State Lands, Marine Facilities Division.**

The following shows actual return times and nominal return times.

Probability of Nonexceedance (%)	Exposure Time (Years)	Return Time (Years)	Nominal Return Time (Years)
50	25	36	50
10	25	237	250
5	25	487	500
3	25	820	800

With mandatory removal from service before 25 year-life

Risk Analysis: In order to insure that the risk levels of a major spill are prudent, a risk analysis determining the levels of hazard and potential for a major spill shall be evaluated and submitted for review and approval.

Peer review: Review of the results of the analysis by an independent peer review panel of experts in structural engineering, geoscience, earthquake engineering, seismic risk analysis, economics, and environmental engineering/science is required. In addition, the findings from the analysis should be reviewed by the public and other stakeholders.

Allowance for Deterioration

In evaluating existing construction, it is most important to:

- Evaluate the actual physical conditions of all structural members to determine the actual sizes and condition of existing members.
- Provide an allowance for corrosion and deterioration.
- Evaluate the properties of the construction materials considering age effects in computing yield strengths. Average actual material properties should be used in the evaluation.
- Evaluate the existing structure details and connections since this is often the weakest link and source of failure.
- Determine the design methodology used by the original designers at the time the structure was designed and constructed.
- Evaluate displacement demands and capacities. Previous code requirements did not emphasize the need for ductility and the failure to include shear and containment reinforcing is most common in existing construction. This has led to numerous structure failures especially when batter piles have been used.

The Appendices of this report present detailed information on underwater inspection criteria and concrete repair.

GROUND FAILURE CRITERIA

Introduction

Ports and marine oil terminals are prone to a variety of geologic hazards. Of these hazards, liquefaction of the saturated, loose cohesionless soils that typically prevail at ports has been the most common source of significant damage, although other hazards – such as direct effects of ground shaking, slope instability, and tsunami – have caused extensive damage as well. Furthermore, experience demonstrates that the seismic performance of soils and port structures is strongly related to the manner in which the fills are placed and improved during construction, and also how the structures are designed and detailed to resist potential geologic hazards.

In an extensive review of the seismic performance of ports, Werner and Hung (1982) concluded that by far the most significant source of earthquake damage to waterfront structures has been pore pressure build up in loose to medium-dense saturated cohesionless soils the prevail in coastal and river environments. This observation has been supported by the occurrence of liquefaction-induced damage at numerous ports in the past decade (Werner, ed., 1998). Components of marine facilities conspicuous for poor seismic performance include: pile supported structures, sheet pile bulkheads, and gravity retaining walls founded on, or backfilled with, loose sandy soils. The generation of excess pore pressures in sandy soils can lead to phenomena associated with the loss of strength of the sandy soils (e.g., loss of bearing capacity, increase in active lateral earth pressure against retaining walls, loss of passive soil resistance below the dredgeline and/or adjacent to anchor systems, excessive settlements and lateral soil movements, buoyancy of buried tanks) contributing to the deformations of waterfront structures. In several instances, the failure of waterfront retaining structures has resulted in significant lateral ground deformations as far as 150 m into backland areas resulting in damage to buildings, tanks and buried utilities.

Sloping ground conditions exist throughout ports as natural and engineered embankments such as river levees, sand or rock dikes, etc., and as dredged channel slopes. Onshore and submarine slopes at ports have been found to be vulnerable to earthquake induced deformations. High water levels and weak foundation soils common at most ports can result in slopes that have marginal static stability and which are very susceptible to earthquake induced failures. In addition to waterfront slopes, several recent cases involving failures of steep, natural slopes along marine terraces located in backland areas have resulted in damage to port facilities. Large scale deformations of these slopes can impede shipping, damage adjacent foundations and buried structures thereby limiting port operations following earthquakes.

In addition to ground failures caused by liquefaction and weak soils marine oil terminals may be vulnerable to additional geologic hazards, as discussed in the appendix (e.g., fault movement and ground displacement, and tsunamis).

Liquefaction Hazards

Methods for evaluating the liquefaction resistance of soils are well documented and relatively simple, straightforward procedures have been adopted for use in engineering practice (Youd and Idriss, 1997). The most common methods of analyses are outlined in the commentary. These methods have been applied over the past two decades in numerous case studies and the strengths and limitations of the techniques have been well established. Although engineering evaluations for the "triggering" of liquefaction are well established, similarly well developed standard-of-practice methods for analyzing the potential consequences of soil liquefaction (i.e., extent of lateral spreading, impact on deep foundations, lifelines and structures) on waterfront components do not exist due to the complexity of these failures.

When evaluating the impact of liquefaction hazards on waterfront components the sensitivity of the structure to permanent deformations must be established. The specification of "performance goals" with respect to soil liquefaction, ground failures and possible mitigation strategies is therefore based on the allowable deformations of structures affected by the liquefaction hazards. From a practical perspective, ground deformations ranging from several inches to several feet represent failure conditions for the broad array of waterfront components at marine oil terminals. The allowable liquefaction-induced deformations of foundation soils will clearly vary with the type of component and ancillary structures, the consequence of failure, and the importance of the component on the post-earthquake operations of the terminal. The level of sophistication required for estimating the liquefaction-induced ground deformations will also vary as a function of the range of tolerable deformations, soil-structure interaction, and the configuration of the component. For example, pseudostatic, rigid body methods may be appropriate for estimating permanent deformations of earth structures affected by liquefaction, however more involved numerical procedures are recommended for liquefaction hazards involving pile supported structures. Along these lines, it should be noted that the factors of safety computed with standard stress-based liquefaction evaluation procedures and pseudo-static design methods are not adequately correlated with ground deformations to facilitate estimates of seismically-induced lateral deformations. For critical and sensitive components numerical analyses which account for the generation of excess pore pressures in foundation soils are recommended.

General Performance Objectives

Design of new structures and upgrade of existing structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas susceptible to severe ground failures if these areas cannot be economically treated with remedial soil improvement.

The presence of potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. The impact of liquefaction hazards on waterfront components shall evaluated in light of allowable deformation limits of the affected components. Since liquefaction is the primary cause of waterfront damage, remediation is a mandatory requirement where the risk of a release of hazardous materials as shown by computation is possible, such as in a pipeline break or tank failure.

Ordinary Construction - Liquefaction associated with construction categorized as "ordinary" shall be evaluated to insure the level of performance is maintained. In general ordinary construction is expected to:

- Resist a moderate level of ground motion without damage;

- Resist a major earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Piers and Wharves- Liquefaction assessments associated with piers and wharves shall be evaluated to insure the level of performance is maintained per the performance goals for piers and wharves stated above.

Essential Construction - Liquefaction evaluation associated with construction categorized as "essential" shall be evaluated to insure the level of performance is maintained. In general essential construction is expected to:

- Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of operation/function and the structural system to remain essentially linear.
- Resist the rare earthquake with a low probability of being exceeded during the life of the structure and operate/function at the level required to meet operational needs.

Hazardous Materials - Liquefaction associated with construction categorized as associated with "hazardous materials" shall be evaluated to insure the level of performance is maintained. In general construction related to containment of hazardous materials is expected to:

- Resist pollution and release of a major spill of hazardous materials for a very rare event

Requisite Ground Motions For Liquefaction Evaluations

The following is based on existing guidelines (e.g., CDMG, 1997; Werner, ed., 1998), current seismic design provisions criteria and appropriate amendments to existing mandates developed for similar facilities. As previously described, the sensitivity and importance of the specific component, as well as the consequence of failure will determine the level of ground motion to be used in seismic design and analysis. The ground motions applied in liquefaction analyses will be selected or generated based on the probabilistic seismic hazard studies in accordance with the appropriate ground motion level (i.e., Level 1, Level 2, Level 3 or Level 4). The ground motions must account for the site specific dynamic response of the soils and represent the motions at depth required for the specific method of analysis.

- **Ordinary category of construction on average seismicity sites**
  For sites of average seismicity, use appropriate code provisions (e.g. NEHRP Provisions contained in FEMA, 1998).

- **Wharves and Piers**
  Design of wharves, wharf dikes, and piers shall use a two-earthquake procedure as shown above in the structural criteria section. Values less than code (NEHRP) are not to be permitted.

- **Essential category of construction**
  Sites where the structure is deemed important and essential shall use a two-earthquake procedure with a Level 1 earthquake and a Level 3 earthquake based on a local site seismicity study. Values less than code are not to be permitted.

- **Construction containing polluting or hazardous material**
  A Level 4 earthquake shall be used.

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and local ground displacement
- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
- Landslides
- Tsunamis

Minimum Acceptable Methods of Analysis

Triggering of Liquefaction The following is taken verbatim from CDMG Special Publication No. 117

"If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location which might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate that those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. *Direct in situ relative density measurements, such as the Standard Penetration Test (ASTM D 1586-92) or the Cone Penetration test (ASTM D 3441-94).*
2. *Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). This is quite straightforward for waterfront sites and groundwater conditions associated with high tide levels should be used in analyses.*
3. *Non-standard penetration test data.*
4. *Geophysical measurements of shear wave velocities.*
5. *"Threshold strain" techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards are low."*

In situations where liquefaction hazards may impact marine facilities (factor of safety less than 1.5), quantitative methods of evaluating the liquefaction resistance of soils are required. The latest consensus pertaining to the evaluation of liquefaction resistance of soils has been presented by Youd and Idriss (1997). The recommended techniques for evaluation are outlined in the commentary.

Lateral Spreading The following is taken verbatim from CDMG Special Publication No. 117

"Lateral spreading on gently sloping ground is generally the most pervasive and damaging type of liquefaction failure (Bartlett and Youd, 1995). Assessment of the potential for lateral spreading and other large site displacement hazards may involve the need to determine the residual undrained strengths of potentially liquefiable soils. If required, this should be done using in-situ SPT or CPT test data (Youd and Idriss, 1997; Seed and Harder, 1990). The use of laboratory testing for this purpose is not recommended, as a number of factors (e.g., sample disturbance, sample densification during reconsolidation prior to undrained shearing, and void ratio redistribution) render laboratory testing a potentially unreliable, and, therefore unconservative basis for assessment of in-situ residual undrained strengths. Assessment of residual strengths of silty or clayey soils may, however, be based on laboratory testing of "undisturbed" samples.

Assessment of potential lateral spread hazards must consider dynamic loading as a potential "driving" force, in addition to gravitational forces. It should again be noted that relatively thin seams of liquefiable material, if continuous over large areas, may serve as significant planes of weakness for translational movements. If prevention of translation or lateral spreading is ascribed to structures providing "edge containment", then the ability of these structures (e.g., berms, dikes, sea walls) to resist failure must also be assessed. Special care should be taken in assessing the containment capabilities

of structures prone to potentially "brittle" modes of failure (e.g., brittle walls which may break, tiebacks which may fail in tension). If a hazard associated with potentially large translational movements is found to exist, then either: (a) suitable recommendations for mitigation of this hazard should be developed, or (b) the proposed "project" should be discontinued.

When suitably sound lateral containment is demonstrated to prevent potential sliding on liquefied layers, then potentially liquefiable zones of finite thickness occurring at depth may be deemed to pose no significant risk beyond the previously defined minimum acceptable level of risk. Suitable criteria upon which to base such as assessment include those proposed by Ishihara (1985).

For information on empirical models that might be appropriate to use in these analyses, see Bartlett and Youd (1995)."

Seismically-Induced Ground Settlement The following is taken verbatim from CDMG Special Publication No. 117

"Settlements for saturated and unsaturated clean sands can be estimated using simplified empirical procedures (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1992). These procedures, developed for relatively clean sandy soils, have been found to provide reasonably reliable settlement estimates for sites not prone to significant lateral spreading.

Any prediction of liquefaction-related, or cyclically-induced, settlements is necessarily approximate, and related hazard assessment and/or development of recommendations for mitigation of such hazard should, accordingly, be performed with suitable conservatism. Similarly, it is very difficult to reliably estimate the amount of localized differential settlement likely to occur as part of the overall predicted settlement: localized differential settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made."

It should be noted that the contractive behavior of sandy soils during cyclic loading is a function of the void ratio of the soil and the in situ stresses acting on the soil. The soil will experience a reduction in volume regardless of its degree of saturation prior to ground shaking, therefore dry sands are also prone to excessive settlements during earthquake loading.

Slope Instability

The most commonly used methods for analysis of slope stability under both static and dynamic conditions are based on standard rigid body mechanics and limit equilibrium concepts that are familiar to most engineers. For use in determining the seismic stability of slopes, limit equilibrium analyses are modified slightly with the addition of a

permanent lateral body force which is the product of a *seismic coefficient* and the mass of the soil bounded by the potential slip. The seismic coefficient (usually designated as k_h, N_h) is specified as a fraction of the peak horizontal acceleration, due to the fact that the lateral inertial force is applied for only a short time interval during transient earthquake loading. Seismic coefficients are commonly specified as roughly $^1/_3$ to $^1/_2$ of the peak horizontal acceleration value (CDMG, 1997).

In most cases involving soils which do not exhibit considerable strength loss after the peak strength has been mobilized, common pseudostatic rigid body methods of evaluation will generally suffice for evaluating the stability of slopes. These methods of evaluation are well established in the technical literature (Kramer, 1996). Although these methods are useful for indicating an approximate level of seismic stability in terms of a factor of safety against failure, they suffer from several potentially important limitations. The primary disadvantages of pseudostatic methods include: (a) they do not indicate the range of slope deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils is incorporated in only a very simplified, "decoupled" manner; (c) progressive deformations that may result due to cyclic loading at stresses less than those required to reduce specific factors of safety to unity are not modeled; (d) strain softening behavior for liquefiable soils or sensitive clays is not directly accounted for; and (e) important aspects of soil-structure interaction are not evaluated.

In most applications involving waterfront slopes and embankments, it is necessary to estimate the permanent slope deformations that may occur in response to the cyclic loading. Allowable deformation limits for slopes will reflect the sensitivity of adjacent structures, foundations and other facilities to these soil movements. Enhancements to traditional pseudostatic limit equilibrium methods of embankment analysis have been developed to estimate embankment deformations for soils which do not lose appreciable strength during earthquake. Rigid body, "sliding block" analyses, which assume the that soil behaves as a rigid, perfectly plastic material, can be used to estimate limited earthquake-induced deformations. The technique, developed by Newmark (1965) is based on simple limit equilibrium stability analysis for determining the critical, or yield, acceleration which is required to bring the factor of safety against sliding for a specified block of soil to unity. The second step involves the introduction of an acceleration time history. When the ground motion acceleration exceeds the critical acceleration (a_{crit}, a_y) the block begins to move down slope. By double integrating the area of the acceleration time history that exceeds a_{crit}, the relative displacement of the block is determined. A simple spreadsheet routine can be used to perform this calculation (Jibson, 1993).

The amount of permanent displacement depends on the maximum magnitude and duration of the earthquake. The ratio of maximum acceleration to yield acceleration of 2.0 will result in block displacements of the order of a few inches for a magnitude 6 1/2 earthquake and several feet for a magnitude 8 earthquake. It should be noted that significant pore pressure increases may be induced by earthquake loading in saturated silts and sands. For these soils a potential exists for a significant strength loss. For dense

saturated sand, significant undrained shear strength can still be mobilized even when residual pore pressure is high. For loose sands, the residual undrained strength which can be mobilized after high pore pressure build-up is very low and is often less than the static undrained shear strength. This may result in flow slides or large ground deformations.

Given that the sliding block analyses are based on limit equilibrium techniques, they suffer from many of the same deficiencies previously noted for pseudostatic analyses. One of the primary limitations with respect to their application for submarine slopes in weak soils is that strain softening behavior is not directly accounted incorporated in the analysis. The sliding block methods have, however, been applied for liquefiable soils by using the post-liquefaction undrained strengths for sandy soils.

In situations where the movement of a slop impacts adjacent structures, such as pile supported structures embedded in dikes, buried lifelines and other soil-structure interaction problems, it is becoming more common to rely on numerical modeling methods to estimate the range of slope deformations which may be induced by design level ground motions (Finn, 1990). The numerical models used for soil-structure interaction problems can be broadly classified based on the techniques that are used to account for the deformations of both the soil and the affected structural element. In many cases the movement of the soil is first computed, then the response of the structure to these deformations is determined. This type of analysis is termed *uncoupled*, in that the computed soil deformations are not affected by the existence of embedded structural components. A common enhancement to this type of uncoupled analysis includes the introduction of an iterative solution scheme which modifies the soil deformations based on the response of the structure so that compatible strains are computed. In a *coupled* type of numerical analysis the deformations of the soil and structural elements are solved concurrently. Two-dimensional numerical models are rather widespread in engineering practice for modeling the seismic performance of waterfront components at ports. Advanced numerical modeling techniques are recommended for soil-structure interaction applications, such as estimating permanent displacements of slopes and embankments with pile supported wharves.

Mitigation of Seismic Hazards Associated with Slope Stability

Remedial strategies for improving the stability of slopes have been well developed for both onshore and submarine slopes. Common techniques for stabilizing slopes include: (a) modifying the geometry of the slope; (b) utilization of berms; (c) soil replacement (key trenches with engineered fill); (d) soil improvement; and (e) structural techniques such as the installation of piles adjacent to the toe of the slope. Constraints imposed by existing structures and facilities, and shipping access will often dictate which of the methods, or combinations of methods, are used.

Requirements for Minimum Allowable Resistance Against Ground Failures

The design of critical structures shall include provisions for the evaluation of potential ground failures. Special care will be given to components such as tank foundations, pipe racks, and buried pipelines to preclude break resulting in release of hazardous materials. Identification of areas prone to geologic hazards is considered a necessary step in the seismic design process. Proper siting of oil terminal components is vital and it is imperative to avoid areas vulnerable to geologic hazards, or areas that cannot be economically treated with remedial ground improvement.

The presence of potentially unstable soils adjacent to oil terminal components (i.e., foundation or backfill soils) shall be fully evaluated for vertical and lateral extent, and expected seismic behavior. Specific attention shall be paid to permanent lateral and vertical ground deformations. At existing facilities, if ground failures are indicated in geotechnical evaluations of sites where the risk of a significant release of hazardous materials would result, then soil remediation, structural retrofit, or re-siting shall be considered.

The seismic performance of waterfront facilities is linked to a large degree by the magnitude of permanent ground displacements adjacent to the component. Therefore structural design provisions must be supplemented with geotechnical criteria for limiting foundation deformations during the design level earthquakes. In the following criteria liquefaction hazards are specifically addressed, however it should be understood that all forms of ground failures must be evaluated in analysis and design. It should also be emphasized that the magnitude of liquefaction induced lateral ground failures are only approximately correlated with factors of safety derived from force, or limit, equilibrium methods of analysis. In light of the fact that these rigid body methods remain the standard of practice limit, the maximum allowable ground deformations for common waterfront components are listed along with minimum factors of safety against liquefaction for foundation soils.

In the following, the allowable ground deformations are a primary design consideration and they shall be evaluated with full consideration of liquefaction hazards.

Note:

The ground deformations and factors of safety in the following sections are presented as target values. These values may be exceeded if it can be shown by reliable procedures that the performance objectives will be met. The ground deformation state must be used with the structural analysis to make certain the structural performance goals and limits are not exceeded.

It should again be noted that within each subset of components the magnitude of the ground deformations causing damage will vary. The following criteria are provided as minimum allowable conditions for insuring the acceptable seismic performance of common structures and waterfront configurations. Unique or sensitive components may require more stringent ground deformation criteria. In addition, the liquefaction criteria

are considered supplementary to the deformation criteria in that the liquefaction criteria can be relaxed if it is demonstrated using appropriate methods of analysis that the deformation criteria have been met for each level of ground motion.

Ordinary Construction Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as structural collapse is avoided.

The following criteria shall be applied for ordinary construction:

Level 1 Earthquake Motions

- Total settlements less than 1 inch.
- Total lateral deformation of less than 3 inches.
- The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

- Total settlements less than 4 inches.
- Total lateral deformation of less than 6 to 12 inches.
- The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground deformations are within the ranges previously specified.

Wharf Dikes Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of appurtenant structures, damage to embedded deep foundations, is avoided and the structure is repairable.

The following criteria shall be applied for wharf dikes:

Level 1 Earthquake Motions

- Total settlements less than 3 inches.
- Total lateral deformation of less than 6 inches.
- The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

- Total settlements less than 6 inches.
- Total lateral deformation of less than 12 inches.

- The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground deformations are within the ranges previously specified.

Gravity Retaining Structures Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of the retaining structures and/or appurtenant components is avoided.

The following criteria shall be applied for gravity retaining structures:

Level 1 Earthquake Motions

- Total settlements less than 3 inches at the top of the wall.
- Total lateral deformation of less than 6 inches at the top of the wall.
- The factor of safety against liquefaction in the foundation and backfill soils shall be greater than 1.5.

Level 2 Earthquake Motions

- Total settlements less than 6 inches at the top of the wall.
- Total lateral deformation of less than 12 inches at the top of the wall.
- The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed wall deformations are within the ranges previously specified.

Anchored Sheetpile Retaining Walls Ground failures due to liquefaction are to be precluded under Level 1 earthquakes. Ground failures inducing limited foundation deformation (i.e., non-flow failures) may occur during a Level 2 earthquake as long as collapse of the retaining structures and/or appurtenant is avoided.

The following criteria shall be applied for anchored sheetpile retaining structures:

Level 1 Earthquake Motions

- Total lateral deformation of less than 4 inches.
- The factor of safety against liquefaction in the foundation and backfill soils shall be greater than 1.5.

Level 2 Earthquake Motions

- Total lateral deformation of less than 10 inches.
- The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed wall deformations are within the ranges previously specified.

Piers and Wharves Under Level 1 earthquake loading unacceptable deformations resulting in widespread damage to the pier and ancillary components (e.g., pipes and utility lines, pavements, conveyance equipment) should be precluded. Structural deformations may occur during a Level 2 earthquake as long as the pier or wharf, and appurtenant components, remains repairable.

The following criteria shall be applied for backfill and foundation soils adjacent to piers and wharves:

Level 1 Earthquake Motions

- Total settlements less than 1 inch.
- Total lateral deformation of the backfill and pier less than 3 inches.
- The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

- Total settlements less than 4 inches.
- Total lateral deformation of the backfill and pier less than 12 inches.
- The recommended factor of safety against liquefaction should be greater than 1.0, however in cases where it may not be possible to achieve a factor of safety greater than 1.0 lower values may be considered as long as the computed ground and structural deformations are within the ranges previously specified.

Essential Construction Under Level 1 earthquake loading deformations resulting in damage to the structure and ancillary components (e.g., pipes and utility lines, pavements, conveyance equipment) shall be precluded. Ground and structural deformations may occur during a Level 2 earthquake as long as they are limited to insure operability of critical functions in the facility. This includes utility lines associated with the structure.

The following criteria shall be applied for backfill and foundation soils adjacent to essential construction:

Level 1 Earthquake Motions

- Total settlements less than 1 inch.
- Total lateral deformation of the foundation and backfill soil less than 3 inches.
- The factor of safety against liquefaction shall be greater than 1.5.

Level 2 Earthquake Motions

- Total ground deformations will be limited to preclude loss of operation and nonrepairable structural damage of the essential component.

Construction containing polluting or hazardous material- Settlements shall be restricted to preclude release of hazardous material causing a major spill. The computed deformation state shall be shown to have limited controlled settlements and restricted lateral spread.

SUPPORTING STRUCTURES AND LIFELINE CRITERIA

Lifelines are facility and utility systems which are vital to the operation of a terminal. They may include electric power, gas and liquid fuels, fire detection and suppression systems, telecommunications, transportation, port operation control facilities, and water supply and sewers. As stated above, safe effective seismic design consists of establishment of performance goals, specification of the earthquake loading, and given that loading, definition of the expected acceptable structural response limits.

When considering a facility/component supporting an essential function, it is critical that the facility/component be considered as a system. It is not sufficient to consider a facility/component simply as a structure or an element, but rather it is required to consider all the elements required to accomplish the objective to be accomplished by that structure or component. This usually includes requirements for electrical power, mechanical systems, water and sewer, communications, road access etc.

Lifeline Performance Objectives

Ordinary Construction / Ordinary Lifelines -Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to

- Resist a moderate level of ground motion without damage;
- Resist a major level of earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Wharves and Piers Lifelines associated with pier or wharves shall be designed with the same levels of service.

- Resist a moderate level of ground motion without damage;
- Resist a major level of earthquake ground motion without collapse, and with the structural in a repairable condition.

Essential Construction / Essential Lifelines - Life line service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:

- Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of operation/function and the structural system to remain essentially linear.
- Resist the rare earthquake with a low probability of being exceeded during the life of the structure and operate/function at the level required to meet operational needs.

Note that essential lifelines can be associated with piers and wharves such as electrical control lines for valves, fire suppression, etc. In such cases the essential lifelines shall be designed to the higher essential category and provision made to account for the deformation state of the pier or wharf on the operation of the lifeline.

Hazardous Materials/Lifelines - Lifeline service associated with construction categorized as containing "hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:

- Resist pollution and release of a major spill of hazardous materials for a very rare event

Provision for tanks and pipelines containing hazardous materials are discussed further below.

Design Earthquakes

The following is based on current criteria and an extension of existing mandates logically applied to analogous situations. Lifeline systems shall be designed to resist the loading produced as follows:

- **Ordinary category of construction on average seismicity sites**
  For sites of average seismicity, use code provisions contained in NEHRP which are based on an earthquake with an approximate 10 percent chance of exceedance in 50 years.

- **Pier or wharf category of construction**
  Sites where the lifeline is associated with a pier or wharf shall use a two-earthquake procedure with Level 1 and a Level 2 based on a local site seismicity study. Values less than NEHRP code are not be permitted

- **Essential category of construction**
  Sites where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 and a Level 3 based on a local site seismicity study. Values less than NEHRP code are not be permitted.

- **Construction containing polluting or hazardous material**
  A Level 4 earthquake shall be used.

Note where essential lifelines are found on piers or wharves and are required for control of hazardous materials, the highest loading shall govern.

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and ground displacement

- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.

- Landslides

- Tsunamis

Modification to Design Ground Motion The ground motions used in design of lifelines may differ from the motions used in conventional facility/structure design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For lifeline component elements located within a structure, the component design loading can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component. In addition, the ground motions used for evaluations of buried lifelines should account for the depth of embedment. If a lifeline is buried at a significant depth (say 10 feet or more) then the ground surface motions should be modified to account for dynamic soil behavior.

Liquefaction And Lifelines

Design of structures shall include provisions to evaluate and resist liquefaction of foundation soils and/or backfill, and account for expected potential settlements and lateral spread deformation. Liquefaction is discussed further in following sections. Liquefaction is the single greatest cause of damage at the waterfront, especially in wharves, quaywalls and retaining structures. Special care must be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of a major spill of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas vulnerable to ground failures such as landslides and lateral spreads. The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected ground deformations (i.e., settlements and lateral earth movements) computed. Since it is rarely possible from an economic or technical perspective to eliminate earthquake induced ground deformations in waterfront environments, specific attention shall be paid to allowable ground deformations . Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformations and stresses. Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe. To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters.

Flexible couplings shall be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components.

- No section of pipe shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.

- Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.

- Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.
- Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.

- Adequate restraints shall be used for all piping.

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a pipe break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax while continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process.

Tanks

All tanks must be anchored. A pattern of well distributed anchor bolts works best compared with fewer larger bolts. A maintenance program is required to inspect the condition of the anchor bolts. Bolts showing corrosion must be replaced. Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. Tank venting is important to restrict implosion.

- Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.

To achieve the required system performance and satisfy regulations, additional hazardous material containment systems are usually used as a backup. Containment systems are composed of either a singular system or a dual system as mandated by public law as discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain will need to be drained.

Design of tanks shall utilize the procedures discussed below.

- Tanks shall be designed against sliding and uplift and be fully anchored.

- Tanks designated as supporting essential functions (such as a fuel tank for a backup generator) shall be designed to resist Level 3 earthquakes using response spectra and the API 650 procedures.

- For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment for a Level 4. This is discussed below in the section Hazardous Materials Containment.

Such spill containment requirements may be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials. This requirement will be discussed below.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Additionally stairways should not be attached to both the foundation and the tank wall.

API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API model may under predict the uplift so a value of twice the API shall be used.

Design of New Tanks The procedures described in American Petroleum Institute Standard 650 (1993 with updates through 1996) shall be used as modified and updated so as not to produce lower loads than what would be required by FEMA 302 Sec 4.1, FEMA (1998).

For essential tanks, response spectra values shall be substituted for equation values. The procedure considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period shall be substituted for ZIC_1. Additionally, sloshing period values shall be substituted for ZIC_2. Tank wall stresses are computed from overturning moments and compared with allowable values. The user shall consider the amount of tank freeboard for sloshing. Failure to provide for sloshing could damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure, and a 0.5 percent damped curve is recommended for sloshing of the liquid.

Evaluation of Existing Tanks Existing tanks shall be evaluated using the procedures for new tank design with an α factor applied determine design earthquake levels. However, when an existing tank is found to be deficient it shall be checked using the procedures described by Manos (1987). Since the new tank design procedures are conservative, an existing tank may be considered as acceptable if it meets the provisions in Manos (1987) and has a lateral acceleration capacity in excess of demand.

Utilities On Piers

Piers may contain pipelines for fire suppression, freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used.

Electric Power

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All equipment shall be anchored as required. Equipment deemed as of ordinary importance shall use lateral force requirements based on provisions of the 1997 Uniform Building Code (ICBO, 1997). Equipment deemed as essential shall have the lateral force requirement computed based on local site conditions using peak ground acceleration for essential facilities (Level 3) and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.

Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and reduce or avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces. Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber.

Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are often not feasible for snubber tie-down since equipment location is variable and may not be defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar; at least 4 snubbers must be used and all snubbers must be rated. Adequate accommodation of differential motion among components must be provided to prevent failure of items like ceramic insulators etc. Adequate cable slack or break away connections must be used.

Telecommunications Lifelines

Telecommunications encompasses conventional telephone requirements, communications and all equipment control lines. The equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants. Some equipment have fragility data. The equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits to prevent overturning.

Traditional damage to telecommunication equipment has included overturning of cabinet mounted electronics, failures of battery racks, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design rules must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points when the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

HAZARDOUS MATERIALS CONTAINMENT

Performance Goal

This section of the criteria is intended to address the seismic design of industrial support facilities, tanks and pipelines which contain hazardous materials. This criteria is intended to produce a level of design such that there is a high probability the facilities and components will perform at satisfactory levels and prevent a release of a major spill of hazardous material throughout their design life. Specifically for industrial support facilities, tanks and pipelines located in areas of high seismicity shall be designed:

- To meet all of the provisions for tanks given above.

- To resist major earthquakes, Level 4, which are considered as very rare events without release of a major spill of hazardous materials.

Design Earthquakes

The industrial support facilities, tanks and pipelines shall be designed to resist the loading produced as follows:

- For sites of average seismicity, use NEHRP provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years or preferably the Level 2 event from a seismicity study if available.

- Where the element/tank is deemed important and essential use a Level 3 earthquake and increase Zone Factor coefficient per response spectra techniques based on a local site seismicity study. Values less than FEMA 302 Sec 4.1 are not be permitted.

- Use a Level 4 earthquake for major spill prevention.

Industrial/Hazardous Tanks and Pipelines Response At Design Loading Levels

Containment systems shall be composed of either a singular system or a dual system as mandated by public law discussed in the Commentary. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems such that a failure will not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which will function should the primary system be damaged.

The structural response of the industrial support facilities, tanks and pipelines under the design earthquake levels shall meet all requirements for nonhazardous material tanks

For a Level 4 earthquake, controlled inelastic behavior with maximum ductility factors to preclude release of a major spill of hazardous materials. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread

deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

ECONOMIC / RISK ANALYSIS

Performance Objective

Marine oil terminal facilities are important facilities. Such facilities represent a huge economic investment by the company operating the facility and at the same time represent a vital resource upon which the State of California and its residents are dependent. An additional issue of pollution of the environment is of major concern. The economic viability of the operation, the need for the resources, and the concern for the environment form a basis upon which to build a framework of decision making. This criteria mandates a safe, design of new facilities and upgrade of old. It must carefully balance the three elements.

There is an increased emphasis on post-earthquake functionality of essential construction. In this light, it is important to be able to evaluate the extent and location of expected structural damage. Are there any weak links in the foundation or structural system design which will preclude operability? Operability demands that the facility be viewed as a total system not just a structural system. Utilities and the other elements must function to have operability. Additionally a procedure is required to evaluate alternative seismic designs/upgrades and select the most effective choice. This guidance presents detailed analysis procedures which can evaluate seismic strengthening, expected damage and the economics and risk of seismic design. The purpose of this procedure is to perform an economic/risk based comparison of alternative designs of a structure considering initial construction expenditures and expected earthquake induced damage over the life of the structure. It may compare different types of construction or different design levels. It is thus intended to assist the user and the design engineer in obtaining cost-effective risk-controlled seismic construction. Chapter 6 of the commentary defines the steps in the procedure for conducting an economic/risk analysis.

The extent to which an existing marine oil terminal needs to be upgraded to enhance seismic resistance depends upon the size of the risk it poses.

There are three possible approaches to seismic upgrade design:

- No Consideration of Risk. Under this option, analysis of potential seismic risks would not be considered in the marine oil terminal criteria; instead, seismic risk reduction would be carried out using a conventional deterministic design or retrofit procedure that presumably meets certain seismic performance requirements under designated seismic hazard levels.

- Risk as a Fallback. Position. Under this option, the user would either upgrade a facility to new construction levels of seismic performance under stringent levels of seismic hazard, or could undertake a risk analysis to justify a lower level of seismic upgrade, as long as the resulting seismic risks are acceptable.

- Total Risk-Based Approach. Under this last option, all seismic risk reduction measures at a marine oil terminal would be risk based; i.e., a seismic risk analysis would be used to check whether the oil terminal system's seismic risks meet certain risk-based criteria..

It should be obvious that design of a seismic upgrade by the first option which does not consider or evaluate the risk could result in expenditures of money while the potential for a large spill may still be unacceptably high. For this reason the second option is suggested as the minimum requirement for design of an upgrade.

Oil Spill Cost and Significance

The cost of an oil spill involves several elements including: the direct cleanup cost involving the expenditures on removal of the oil, the loss of use factors, the cost of damage to the coastline and the environment in the form of the destruction of wild life and natural resources. Additionally there are third-party damages consisting of individuals who suffered property damage from contact with the oil.

The State of California Office of Oil Spill Prevention and Response estimates the cost of an oil spill based on an average of 108 oil spill incidents as follows:

Cleanup cost	$150 /gallon
Third-part cost	$100 /gallon
Natural resource damage	$200 /gallon
Total Cost	$450/gallon

Noting that there are 42 gallons per barrel, the cost of a 1200-barrel spill would be $22,680,000. The 1990 Oil Pollution Act establishes a level of financial responsibility for a 1000-barrel oil spill in federal waters at $35 million. It is obvious that a 1200-barrel spill is a very large and costly event. The size of a potential spill and the associated costs must be included in a risk analysis.

Outline of Risk Analysis Procedure

The risk approach is described in more detail in the Chapter 6. The major steps of the procedure, as they are given in Chapter 6, are summarized:

(1) Define system and components to be evaluated;

(2) Identify seismic risk reduction alternatives;

(3) Define multiple scenario earthquakes;

(4) Estimate site-specific seismic hazards;

(5) Implement alternative seismic design/strengthening strategies for individual components within overall system;

(6) Evaluate seismic performance of overall system; and

(7) Assess seismic risks and modify component designs if appropriate.

The specific substeps under Step 7 are summarized,

(7-1) Develop risk and decision calculations for risk reduction alternatives;

(7-2) Select risk reduction alternatives that best fit performance criteria; and

(7-3) Review selections of risk reduction alternatives with public.

The results of a economic/risk analysis are expressed in the form of cost vs. risk. The study not only shows the economics of the decision making process of selecting alternative designs, but also gives insight of component behavior showing which elements form the "weak links". The analysis quantifies the reduction in spill potential for various upgrade options. Thus the effectiveness of the economic investment for each upgrade alternative can be shown in terms of the risk of a major spill.

Supporting lifelines are part of the overall marine oil terminal system that need to be considered when evaluating whether the levels of risk to life safety, the environment, terminal operations, or economic losses are acceptable. Performance requirements given for supporting lifelines should be consistent with overall performance requirements given earlier.

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CHAPTER 1 CHARACTERIZATION OF REGIONAL SEISMICITY AND GROUND SHAKING HAZARDS

Introduction

The objective of this chapter is to introduce the seismic and geologic hazards that have contributed to severe damage at port facilities during recent earthquakes, as well as provide an overview of the methods that are commonly used in practice to evaluate these hazards. This chapter is intended to serve as a resource document and as such, the introductory material is supplemented with numerous references to assist interested readers in locating additional sources of practical information on hazard analyses for ports. In addition to the references that address the specific topics discussed herein, the reader is directed to several recent publications which provide comprehensive treatment of seismic and geologic hazards (e.g., Kramer, 1996; Okamoto, 1984) and seismic design issues for port facilities (e.g., Port of Los Angeles, 1990; Tsinker, 1997). The recent report "Guidelines for Evaluating and Mitigating Seismic Hazards in California" prepared by the CDMG (1997) is a particularly worthwhile reference.

A comprehensive seismic hazard evaluation addresses topics such as the spatial and temporal occurrence of earthquakes, the characteristics of the ground motions that may be anticipated over specified time intervals, and the dynamic response of soils subjected to the design level ground motions. These factors collectively define the seismic hazard at a site. The evaluation of these hazards will generally proceed in five primary steps that include: (a) identifying potentially active seismic sources in the region, (b) estimating the seismicity associated with the individual sources, (c) evaluating the influence that the travel path has on the characteristics of the seismic waves as they propagate from the source to the site, (d) assessing the dynamic response of near surface soils (addressed herein), and finally (e) analyses of the stability of foundation soils and structures subjected to the design level ground motions. The collection and synthesis of this information involves input from geology, seismology, and engineering disciplines.

Seismic Source Identification and Characterization

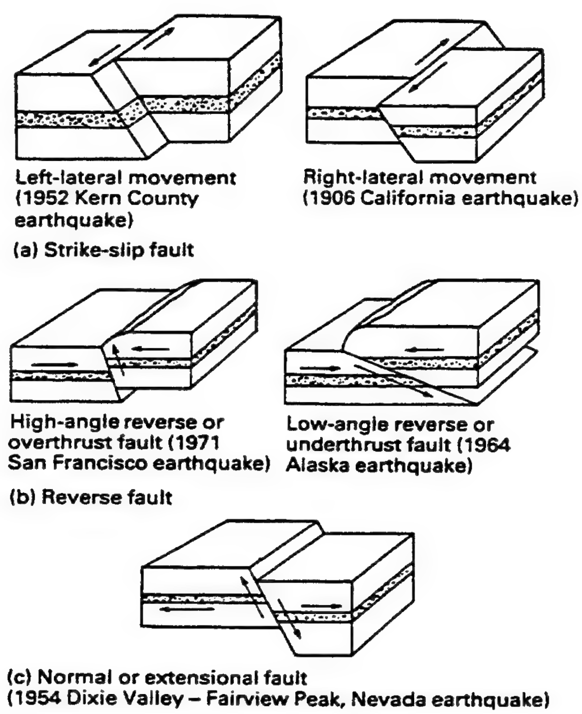
The identification of seismic sources in the region of interest and the evaluation of the seismicity attributed to these sources forms the basis for the seismic hazard analysis. Primary seismological issues include: the location of local earthquake sources, the rate of seismicity attributed to each source as a function of earthquake magnitude, and the maximum credible earthquakes that could be generated by each seismic source. This evaluation requires the synthesis of geographic, geologic, and seismological data. An outline of the major components of a seismic hazard evaluation is provided in the following sections.

Characterization of the Regional Tectonic Setting

Earthquakes are associated with the release of strain energy along discontinuities in the earth's crust. These discontinuities, or faults, are manifested as crustal plate boundaries and

fractures within plates. Most of the world's seismicity occurs along the plate boundaries due to the relative motions of the adjacent plates. Significant earthquakes have, however, also occurred within the crustal plates (termed *intraplate* earthquakes) along ancient rift zones and in regions of volcanic activity. The style of faulting, rate of seismicity, and the size of the greatest earthquakes associated with a potential seismic source are related to the tectonic processes in that region and the resulting stress patterns in the shallow crust. Excellent introductions to the seismological aspects of seismic hazards (i.e., global tectonics, the causes of earthquakes, earthquake magnitude scales and related topics) have been presented by Bolt (1993) and Idriss (1985).

The progressive increase in crustal stresses leads to the failure of crustal rocks along faults. The direction of the fault rupture is used to characterize various fault types. Three general types of fault - strike-slip, reverse, and normal - are illustrated in Figure 1-1. Pure examples of these fault types rarely occur; rather, the relative movement along the fault has components both parallel and normal to the fault trace. It is important to characterize the pattern of crustal stresses in that this determines the type and depth of faulting, and influences hazards such as surface faulting, enhancement of ground motions due to the rupture mechanism, and directivity of potential seismic energy release.



b) Fault types

Figure 1-1: Fault Types (Werner, 1991)

A variety of tectonic regimes exist in California. The tectonic provinces in the United States can be generally classified based on the mechanisms that produce the earthquakes and the style of faulting that is observed. For example, (a) subduction zones found in the coastal regions of northernmost California (north of Cape Mendocino) lead to thrust-type earthquakes (e.g., M_s 7.0 1992 Cape Mendocino earthquakes); (b) transform faulting along the western margins of

California produce predominantly strike-slip earthquakes (e.g., M_s 8.2 1906 San Francisco earthquake, M_w 6.9 1989 Loma Prieta earthquake); (c) intraplate rift zones and Basin and Range faulting have produced large historic earthquakes in the eastern portion of California; and (d) volcanic processes have also produced significant earthquakes. With respect to marine oil terminals located in coastal regions and along inland waterways the seismic hazard is associated with the first two types of faulting.

Local variations in the crustal stress fields and fault geometry within broad regional tectonic regimes can result in earthquakes of different rupture process in the same region (e.g., M_s 6.4 San Fernando and M_w 6.7 1994 Northridge thrust-type earthquakes). In this the symbols M_s and M_w denote the surface wave magnitude and the moment magnitude of the earthquakes, respectively. These represent two of several magnitude scales that have been developed by seismologists (Bolt, 1993; Idriss, 1985). The surface wave magnitude and moment magnitude are the most commonly referenced magnitude scales in engineering practice and the difference in the magnitudes is minor in the range of practical interest (M 6 to 8).

The identification of regional faults, fault systems, or seismic source zones is the first step in the seismic hazard evaluation. The location of seismic sources is based on contributions from historic observations, surface mapping of offset strata, surface geomorphology, trenching studies, geophysical imaging, aerial photo interpretation, remote sensing, and geodetic leveling.

With respect to the historic record of earthquakes, the US Geological Survey National Earthquake Information Center, Denver, Colorado has produced EPIC, "The Global Hypocenter Data Base" (NEIC, 1992), a CDROM which contains parameters for more than 438,000 earthquake events. Seven world-wide and 12 regional earthquake catalogs were assembled to produce this data base, spanning a time period from 2100 BC through 1990. Useful data for the United States is generally constrained to the period when instruments were available to compute event magnitude. Each earthquake is detailed where data are available with date, origin time, location, magnitude estimates, intensity, and cultural effects.

A computer program, EPIC, is available for searching the CDROM. EPIC makes data available to information users via a user-defined search request. The request determines which steps are necessary to produce the desired output. An automated plotting package that produces seismicity maps in multicolor or monochrome is incorporated into the EPIC software. The data to be mapped are extracted from the selected data and plotted in a global or regional format. The availability of the CDROM data base of epicenters and EPIC software greatly facilitates creation of the historical epicenter subset required for use with automated site seismicity analysis tools. Details are presented in the EPIC user's manual, which will not be repeated here.

A number of data fields for some events are unfilled because the information is not available. Information on cultural effects, intensity, and other phenomena associated with the event has been included for earthquakes in the United States. The quality of epicenter determinations varies significantly with the time period studied. Before 1900, locations are usually noninstrumentally determined and are given as the center of the macroseismic effects. Most instrumental epicenters prior to 1961, excluding local earthquakes in California, were located to the nearest 1/4 or 1/2 degree of latitude and longitude. Reliable information on the

quality of many epicenter determinations is lacking. Beginning in 1960, epicenters have been determined by computer, and the accuracy is generally better. However, although stated to tenths or hundredths of a degree, the location accuracy is usually a few tenths of a degree. Since May 1968, the latitude and longitude values for most events have been listed to three decimal places. This precision is not intended to reflect the accuracy of the location of events except for local California earthquakes and special epicenter determinations. Where several sources have determined an epicenter for the same earthquake, one solution has been designated as the most reliable. Usually it is the source believed to contain the best data set for the earthquake. In some cases, data from two sources were combined to provide a more complete record. Magnitudes from a number of different sources are included in the earthquake data file. Gutenberg and Richter (1954) and Richter (1958) discuss the development of the magnitude scale. Many magnitudes published by Gutenberg and Richter (1954) were later revised by Richter (1958). The revised magnitudes are used in the file even though the source is identified as Gutenberg and Richter (1954). The concept of earthquake magnitude is not restricted to one value. Several definitions are possible, depending on which seismic waves are measured. Three different magnitude scales, body wave (m_b), surface wave (M_s), and local (M_l), are distinguished in this file. In addition, another data field, other_magnitude , was included when it was unclear which scale was used. Recent earthquakes are being defined by moment magnitude. Richter (1958) and other modern seismology references provide detailed discussions on the topic of magnitude determination. The different scales do not give exactly comparable results, and different values frequently are given for the same earthquake (Idriss, 1985). It is common practice to average the individual magnitudes from different stations to get a more uniform value within each scale.

In general, the file contains earthquakes of magnitude less than 4.0 only for the United States region and for areas within dense seismic station networks. However, no claim is made for the statistical homogeneity of these events. Inclusion of earthquakes of magnitude 4.0 to 5.0 also is influenced by the proximity of seismic stations to the source or epicenter. A maximum intensity is listed for many of the earthquakes. Each is assigned according to the Modified Mercalli Intensity Scale of 1931 (Bolt, 1993). Some of these values have been converted from reported intensities on other scales.

A period of demonstrated quiescence over a geological time period indicates inactivity of the fault and probable continued inactivity. However, inactivity over a period of historic recording (50 to 100 years) does not imply future inactivity. Rather, it may point to a region which is locked and through which a major fault rupture may propagate. A number of earthquakes producing damage in southern California occurred on faults lacking historic activity. Caution must be exercised to recognize that the limitations of an incomplete data base when extrapolating to return periods greatly exceeding the length of the period of recorded data. Furthermore, aftershocks must be distinguished from main shocks. An area having recently undergone a large event releasing strain built up for hundreds or thousands of years is probably safe against a large release in the near future. Thus, a recent large event on a fault might actually indicate safety in the immediate future, rather than an indication of increased activity. A single event by itself cannot give an accurate measure of return time.

Despite the tremendous advances in fault identification that have been made possible by geophysical imaging studies, field mapping, and deep drilling projects, uncertainties still exist in

the identification of faults capable of generating damaging earthquakes. Faults may lack surface expression due to burial under thick sedimentary deposits, or the combination of very low deformation rates and active geologic processes such as erosion which obliterate evidence of faulting. Several notable examples of concealed seismic sources include: "blind" thrust faults (M_w 6.7 1994 Northridge earthquake), rupture along folded strata at depth (M_s 6.7 1983 Coalinga, CA earthquake). Of particular concern may be the existence of potentially active off-shore seismic sources that have not been well characterized. Once faults have been identified, the seismicity associated with the feature must be assessed (as discussed below). Detailed maps that identify active faults are available in only a few areas of the United States. One such region is the San Francisco Bay area in California, Figure 1-2. The network of faults associated with the San Andreas Fault System are relatively well defined in this region, and distances between the site of interest and the local faults are well constrained.

In several regions of California the tectonic processes responsible for historic earthquakes have not been well defined. This is particularly true in regions of diffuse and low level seismicity such as the Central Valley (Sacramento-San Joaquin). Although these regions have been characterized as exhibiting relatively low seismicity, notable earthquakes have occurred (e.g., MMI IX 1892 Vacaville earthquake, MMI IX 1892 Winters earthquake)). In these regions the tectonic provinces are established from the geologic history of the region, structural trends in geologic units, geographic terrain, and measured crustal movements from geodetic investigations.

Evaluation of Potential Seismic Sources

The second step in a seismic hazard analysis incorporates historic seismicity, geologic evidence for prehistoric earthquake activity (termed *paleoseismicity*), and comparisons with similar tectonic regions around the world in order to establish the seismicity of the regional seismic sources. At this stage of evaluation the rate of seismicity (i.e., the recurrence interval for probable, earthquake are established. The historic record has been earthquakes of various magnitudes) and the estimation of the maximum credible, or used as one of the primary indicators

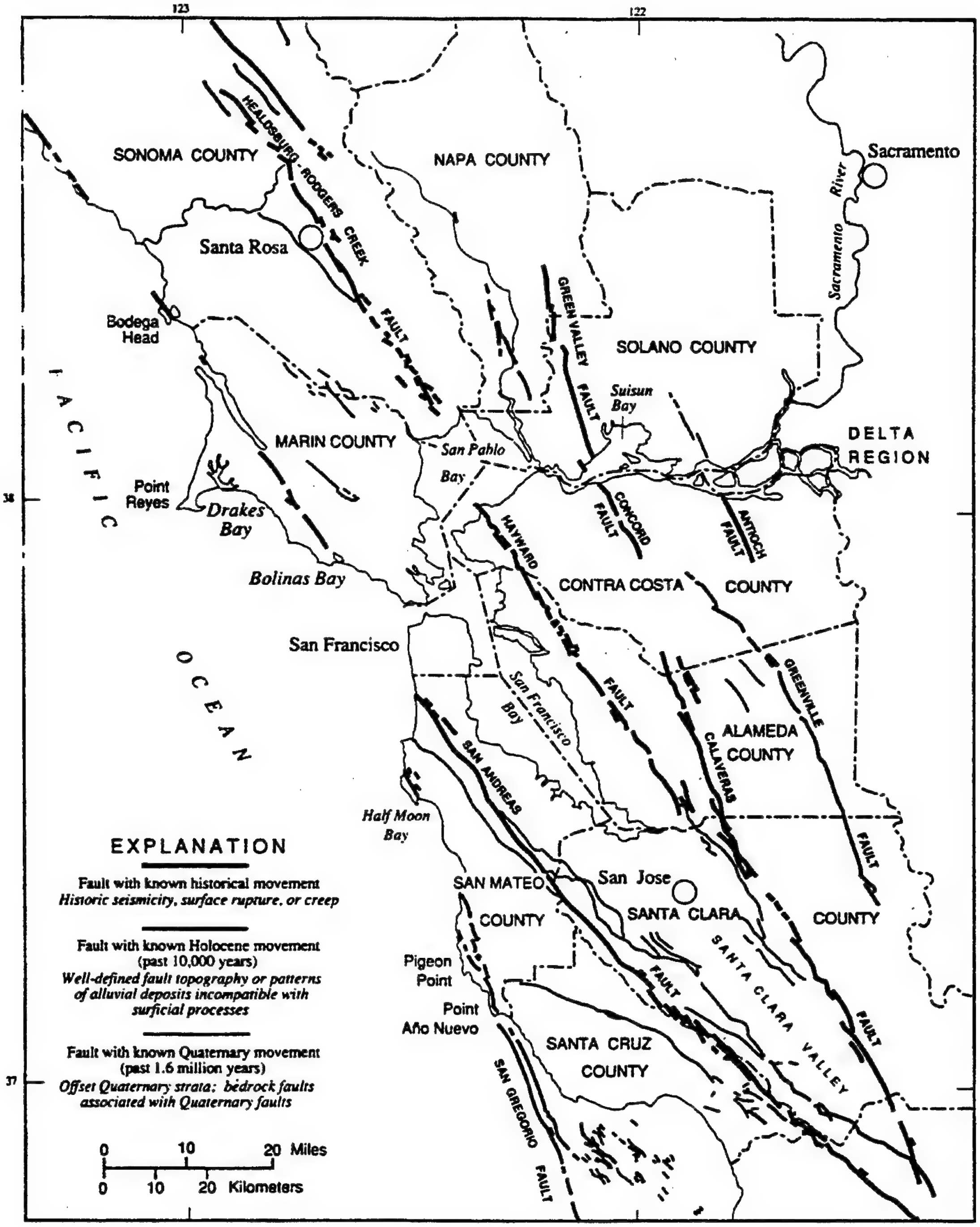


Figure 1-2: Faults In The San Francisco Bay Region (Modified From Brown And Kockelman, 1983)

of earthquake activity in most regions of the United States. The plate boundaries are well defined along the western United States by the historical patterns of seismicity. It is evident that although the highest rates of seismicity are found along coastal California, potentially damaging earthquakes have occurred during the historic record in many regions of the state. Several of these occurred in coastal regions and along inland waterways in other portions of the state.

The historic record of earthquakes in most of the United States is relatively short when compared to the length of time that would be required to accurately assess, from a statistical standpoint, the "average" rate of seismicity in a region. For this reason the historic record of earthquakes should be viewed as an incomplete indicator of seismicity levels in a region. In this situation the historic record of earthquakes must be supplemented with indirect lines of geologic evidence for faulting. Geologic investigations include: geomorphology studies of ground surface features along faults (e.g., sag ponds, offset streams, rift valleys), fault trenching studies, paleoseismicity investigations which look for evidence of ground failures caused by prehistoric earthquakes, and geophysical investigations to detect the deformation of soil and rock units at depth. From an engineering perspective, a fault is considered "active" if evidence has been found for earthquakes during the Holocene Epoch (i.e., the last 11,000 years).

The hazard posed by a potential seismic source is directly related to both the size of the earthquakes generated along the fault and the recurrence interval for damaging earthquakes. Given the relatively short historic record of earthquakes in most regions of the United States, it is doubtful that the "true" pattern of seismicity has been established in most regions. As a result of the scarcity of long-term seismicity data, the maximum credible earthquake (MCE) that can be attributed to a potential seismic source is usually specified independent of the time (i.e., deterministically) on the basis of geometry of the fault and the tectonic setting. The rate of seismicity is, by definition, based on the recurrence intervals between earthquakes of various magnitudes. Establishing the recurrence intervals for earthquakes will reflect the historic rate of seismicity, as well as the length of the historic record of earthquakes. In most regions of the United States, the combination of a relatively short period of observation (200 to 400 years) and low to moderate seismicity requires that the rate of seismicity used as the basis for engineering design be based on probabilistic methods of analysis. Useful overviews on the characterization of seismic sources have been prepared by Cluff and Coppersmith (1990), Coppersmith (1991) and Power et al. (1986).

Maximum Earthquake Magnitude

Once the potential seismic sources in a region have been identified, the maximum earthquake magnitude is estimated from historical seismicity and/or geologic data. Methods which are used to estimate the largest earthquake that may be generated by a specific source without regard to the length of time that may elapse between earthquakes of this size are termed deterministic. By specifying the MCE as independent of time, the worst case scenario is established. Empirical techniques which relate fault geometry to the magnitude of the largest earthquake are commonly used in deterministic analyses. One such method which relates measured surface rupture lengths to the moment magnitudes of the causative earthquakes is shown in Figure 1-3. It is common for practitioners to use relationships such as this to estimate the maximum credible earthquake for faults based on the mapped length of the fault. For example, this technique could be applied to the faults illustrated in Figure 1-2. While this

versus magnitude in the form shown in Figure 1-4. The equation for the line on this semi-logarithmic plot is termed the *Gutenberg-Richter equation* and it has the form:

$$\log_{10} N = a - bM \qquad 1\text{-}1$$

where N is the number of shocks per year for a given magnitude (M), 10^a is the mean yearly number of earthquakes of magnitude greater than or equal to zero, and b describes the relative likelihood of large and small earthquakes. Mathematically, the b-value is the slope of the logN versus M line and it is widely used to model regional seismicity rates.

The b-value is important in that it represents the rate of seismicity for the region. Significant uncertainty can exist in specifying the b-value since the line is not well constrained for small magnitudes due to limitations in earthquake detection, and more importantly, for the larger magnitudes due to the incomplete record of earthquakes and the relatively small number of large earthquakes. The plot is, however, very useful in demonstrating that as the specified return period, or "exposure", increases the size of the largest earthquake that is likely to occur during that span of time also increases. This increase in magnitude is not a linear function of time. Furthermore, the anticipated earthquake magnitude does not continue to increase as the recurrence interval increases. The maximum earthquake will be limited to the maximum credible earthquake established using deterministic methods of evaluation.

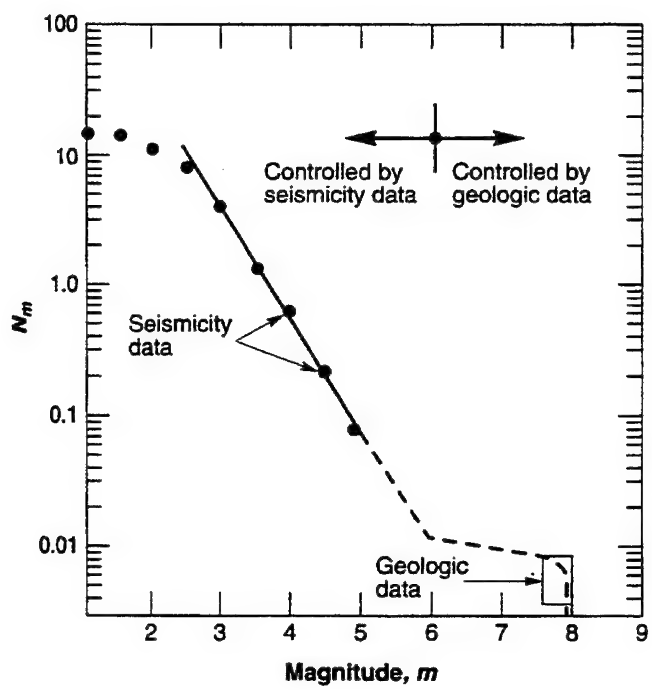


Figure 1-4: Inconsistency Of Mean Annual Rate Of Exceedance As Determined From Seismicity Data And Geologic Data (Youngs And Coppersmith, 1985)

Geologic Data It is very difficult to establish the Gutenberg-Richter frequency-magnitude relationship for individual faults. Given this shortcoming, an alternative method that is based on the average slip rate along the fault is used to supplement the historic record of earthquakes for an individual source. Slip rates obtained from geomorphology studies, fault trenching,

geophysical investigations, or estimates made by comparisons to other faults in similar tectonic regimes are related to the moment magnitudes (proportional to the area of the fault rupture times the average fault displacement, or slip, during the earthquake) for credible earthquakes. Long term slip rates can be used to estimate the average rate of seismic moment release from a fault. Using relationships between earthquake magnitude and seismic moment one can then use the fault slip rate to estimate the rate of earthquake occurrence on the fault (Geomatrix, 1995).

Seismic hazards in a region are rarely due to a single seismic source. In most seismically active regions several faults or a network of closely spaced faults contribute to the overall seismic hazard. The maximum credible earthquake and seismicity rates will vary for each source. The seismic hazard at a port facility in these regions will reflect the combined hazard from each of the individual sources. In light of the fact that the recurrence intervals for damaging earthquakes generated along each fault vary, the hazard must incorporate the contribution of each of the faults for the return period of interest. The synthesis of the seismicity data for each source in the evaluation of the seismic hazard for a specific site is usually performed using probabilistic analysis methods.

Probabilistic seismic hazard analyses are commonly used to evaluate the relative contributions of each seismic source on the overall seismic hazard. Probabilistic methods offer several distinct advantages in establishing the contribution of the individual seismic sources on the aggregate hazard at a site. These include: (a) varying rates of seismicity can be incorporated, (b) the influence of uncertainties in the source characterization on the resulting design level motion can be evaluated, and (c) the influence of return period on the anticipated intensity of ground shaking can be determined. These factors are assessed through the use of "logic-trees" which allow systematic consideration of uncertainty in the values of the parameters of a particular seismic hazard model. Introductions to the concepts behind probabilistic analyses and their practical application are contained in Coppersmith (1991), Kramer (1996), Power et al. (1994), and the Working Group on California Earthquake Probabilities (1990).

The use of logic trees facilitates the incorporation of alternative hazard models in evaluating the seismic hazard at a site. As shown in Figure 1-5, each of the different models are assigned a weighting factor which indicates the relative reliability of the parameters being used in the model. The logic tree in Figure 1-5 was developed during a seismic source characterization study of the. The headings list various aspects of the fault geometry that affect the recurrence relationships for subduction earthquakes along the interface between the Juan de Fuca and North American plates. The weighting factors are given in parentheses and these represent the relative confidence that the investigators had in the individual models. It should be noted that the specific weighting values used are subjectively determined from empirical data on the characteristics of faults in similar tectonic environments and considerable judgment.

A Procedure For Computing Site Seismicity (U.S. Navy Method, Ferritto,1994)

As noted by Coppersmith (1991), many elements of seismic source characterization depend on the tectonic environment. A seismic model must be based on the knowledge of the

local area. It can consist of either an area source zone or a detailed fault definition region. Where specific faults are identified as contributing to the regional seismic hazard these sources can be modeled as a line source extending along prescribed portions of the fault. Where a fault exhibits variations of activity along its length, it can be divided into subelements containing regions where activity is uniform.

For the procedures developed herein, a fault segment can be modeled by two line segments defined by three points. The events to include or associate with the fault are defined by specification of a distance from the fault line, such that all those events within the distance are grouped with the fault. Alternatively, a region can be designated by four points to bound the fault. Again, note a fault can be divided into pieces where activity or geometry so dictates.

Source zones are specified as regions where a zone of like seismicity is evident. The regional geology and tectonics assist in defining the source zone boundaries. A source zone is defined as a region of uniform seismicity, such that an event is equally likely to occur in any portion of the zone. This is characterized by the concept of a "floating earthquake," an event that can occur anywhere in the zone.

In the development of a site model, it is important to keep in mind that an equivalent representation of a region is being created by a series of fault line segments or source zones. The seismicity must be captured in terms of its spatial location and in terms of the level of activity. Assignment of events to one fault source as opposed to another increases that fault's contribution to the estimation of event recurrence. It is important to capture all of the regional the seismicity. For faulting conditions where there are a number of parallel elements, it may not be easy to separate which events are associated with which fault. Consideration must be given to the dip of the fault in assigning events, since the epicenter for a sloping fault can actually occur in a number of kilometers away from the surface trace of the fault. The large majority of strike slip faults have steep dips of 70 degrees or greater. On the other hand, thrust faults generally have dips much less than this, generally in the range of 45 to 60 degrees. For the cases where a fault is close to a site (within 10 miles), special considerations should be given to the location of the fault line segments that define the fault model. If the fault dips toward the site, the actual epicentral distance may be closer to the site than the surface trace of the fault. For faults at greater distances, the difference becomes less significant.

Once a fault or region has been defined as a seismic source, the maximum earthquake magnitude must be defined. In a previous section, a plot was shown relating fault rupture length to magnitude. The length of a fault can be estimated from maps. An assumption can be made that a fault will rupture over 50 to 80 percent of its length. This estimate of rupture distance can be used to define the fault magnitude. Estimates of fault magnitudes have been made for some Western United States faults. It is essential to review previous geologic and seismological studies for the region to develop an understanding of the site's tectonic setting and seismic potentials.

Computation Of Recurrence Parameters

The procedures discussed in this section are equally applicable to regional analysis or fault analysis. The subset of events assigned to the source zone of interest are used to calculate

the Richter A and B coefficients, Equation 1-1 above. This computation defines the earthquake recurrence as a line on a semilog plot. The linear segment is bounded by a maximum magnitude determined as discussed above and by a minimum magnitude below which the data becomes nonlinear. Typically, the value of B is about -0.9. The general earthquake recurrence is thus initially defined. However, as will be shown in the following sections, two important elements are added to geologic slip data and characteristic magnitude.

Geologic Slip-Based Recurrence

A procedure were presented above for calculating recurrence based on the geologic slip rate data. Once the seismicity is estimated from the historical data, the geologic data can be compared. The procedure allows the user to adjust the A and B values from the historical data to include information based on the longer span geologic data. Should other studies be available, the results of these individual fault studies can be used here by adjusting the recurrence parameters.

Characteristic Magnitude

As discussed above, geologic data may show the presence of history of a characteristic event at some average return time. The seismicity defined by the historical data fails to capture this activity, so it is important to include it within the set of events developed for the fault. Once the size of the event and the effective average return time is defined, it is possible to include this in the analysis. Again, if studies with more advanced models are available to define temporal distributions, that data can be used here.

Computational Procedure

Various approaches were presented above to determine the probability of earthquake occurrence. As shown above, various amounts of data are required, some of which are beyond the scope of an engineering investigation. The engineer is free to use any documented procedure which will achieve valid results.

One approach was taken in the formulation of a Monte Carlo simulation procedure, Ferritto (1994). The procedure uses the fault model and regional model discussed earlier, together with the recurrence procedure. As stated above, the A and B parameters combined with geologic slip rate data and characteristic magnitude form the basis for the recurrence function. Once the recurrence function for a fault is defined, the magnitude distribution can be computed. The process is done for each fault individually. A list of 5,000 events representing the largest magnitudes expected to occur in 50,000 years is computed. For each magnitude, a fault break length is determined using data by Coppersmith (1991). A random epicenter location is selected along the fault. The fault break is then assigned to the random epicenter. Various distances are computed, such as epicentral distance, hypocentral distance, and closest distance of fault break to site. The choice of distance depends on the acceleration attenuation equation chosen by the user.

Using the magnitude and separation distance, a site acceleration and standard deviation are computed. A random acceleration is then determined. Associated with each acceleration is the causative event and distance. The process is repeated 5,000 times for each fault. The random fault data are then combined for a total site probability distribution. The procedure described above has the advantage that historical data are augmented with available geologic slip data. Where characteristic events are defined, they may be easily incorporated at the appropriate return time. The effective nonlinear recurrence function attempts to capture the temporal characteristics of the data without complex estimates of Markov or Bayesian parameters.

Crustal Deformation Hazards

The process of fault rupture and the release of strain energy results in permanent crustal deformations. The surface manifestation of this deformation will reflect factors such as the type of faulting (thrust versus strike slip), the magnitude of the earthquake, and the nature of the near-surface rock or soil (Bray et al., 1994). These seismic hazards include relatively deep seated crustal deformation, and surficial deformations such as ground rupture, and creep along the fault. Deep seated deformations tend to be very broad and regional in nature while surficial deformations are generally very localized along faults. The seismic hazards associated with tectonic deformations can be generally confined to (a) regional vertical deformations in regions of

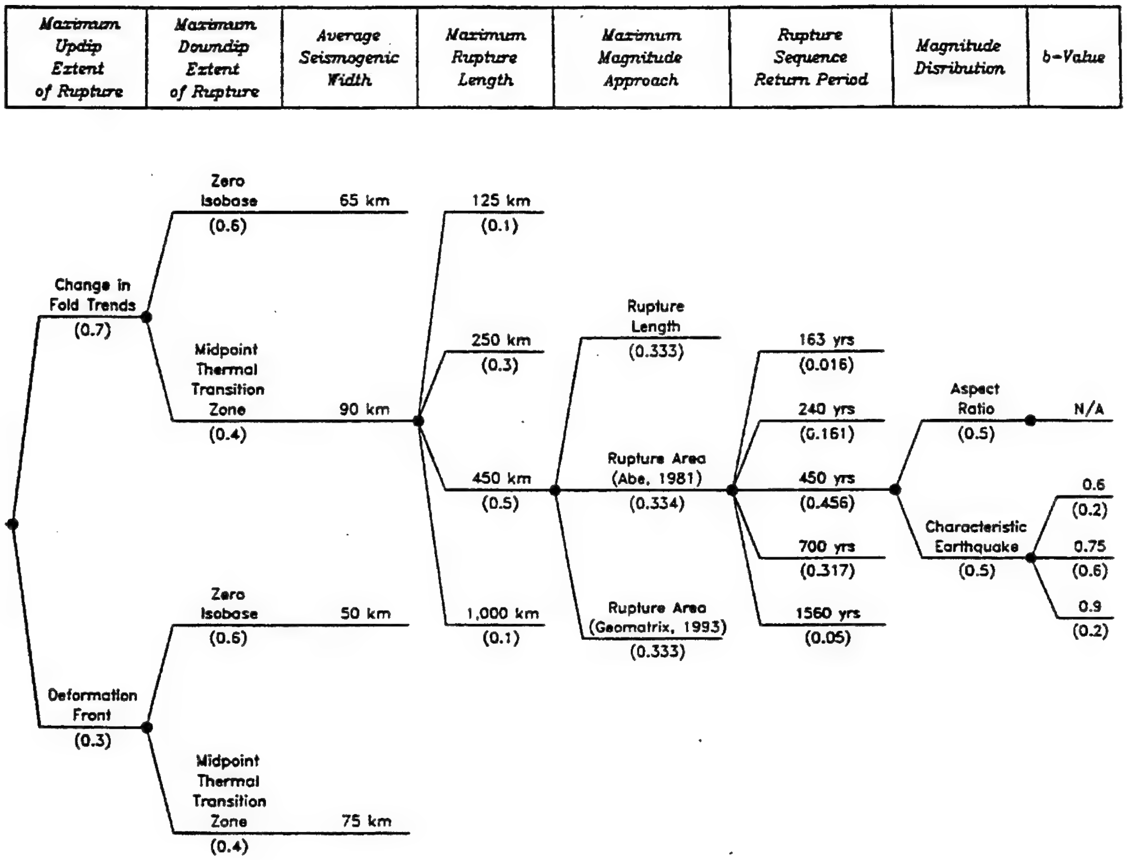


Figure 1-5: Seismic Source Characterization Logic Tree For Cascadia Interface Source (Geomatrix, 1995)

dip-slip faults (thrust or extensional faulting), and (b) surface deformations which occur along, or in close proximity, to faults that rupture at the ground surface. The first of these scenarios may affect regions with subduction zone earthquakes while the second phenomena will be of concern to ports that are located on active faults.

Ground Surface Rupture Ground surface rupture is commonly associated with earthquakes of magnitude 5.5 and greater. The extent of the rupture and the displacement across the fault generally increases with magnitude. Average displacements across the faults vary from approximately 1 cm for M_w 6.0 to as much as 7.5 meters for M_w 8. Experience during earthquakes demonstrates that lifelines (e.g., water, power, transportation and communication lines) that cross active faults are vulnerable to damage from fault offset. Although the likelihood of fault passing directly through a port is small, port authorities should be aware of the lifelines that serve the port, and the routing of these lifelines relative to local faults.

Creep Slow, aseismic crustal movements across faults are termed creep. Although this phenomenon is not associated with strong ground motions, it does constitute a seismic hazard in

several regions of the United States. As with earthquake induced displacements across faults, long-term creep affects lifelines that cross the fault by producing relative offsets.

Ground Shaking Hazards

This section provides background information on earthquake ground motions, including the characterization of strong ground motions for engineering design purposes, the geological parameters that affect the strength of the ground shaking, and current methods for estimating site-specific ground motion parameters. The behavior of soil deposits and structures during earthquakes is dependent on the strength, frequency content, and duration of ground shaking. The strength and duration of the ground motions are fundamentally related to the seismic energy imparted into a body, while the frequency content is important in assessing the response of structures. Procedures for characterizing each of these parameters are summarized below.

Characterization of Ground Motions

For earthquake engineering of structures and soil materials, potential ground motions at a site are characterized in terms of their strength, frequency content, and duration. These characteristics are discussed below. Further discussion of these ground motion characteristics with regard to their use in the seismic design and analysis of port structures is provided in following chapters.

Strength of Earthquake Ground Motions

Qualitative Measures The strength of earthquake ground shaking has historically been characterized on the basis of qualitative intensity estimates and, with the advent of strong motion recording instruments, peak acceleration and other quantitative single-parameter measures of the strength of the shaking as obtained from recorded ground motions. The intensity of the ground motions is established using a qualitative scale that uses Roman numerals to represent the effect of the earthquake shaking; (a) on persons in the felt area (i.e., human perception of the ground motions); (b) the response of structures and other objects; and (c) the ground (i.e., geologic phenomena induced by the earthquake shaking). One such scale -- the Modified Mercalli Intensity scale -- has been widely used in North America as a method of representing the strength of earthquake motions in the absence of recorded motions. Other qualitative intensity scales are used in other parts of the world. Maps which illustrate the relative intensity levels in affected areas (termed isoseismal maps) have been used to estimate ground motion parameters for use in engineering analysis and design in regions lacking instrumental strong motion data (Bolt, 1993; Kramer 1996).

Quantitative Measures It is common in engineering design and analysis to characterize the strength of the ground shaking using simple, single-parameter, quantitative measures, such as peak acceleration, velocity, and displacement; and effective peak acceleration, velocity, and displacement). Such parameters have gained wide acceptance because they are easily incorporated into standard pseudostatic methods of analysis. However, the characterization of a

transient time history of motion using a single peak ground motion amplitude fails to account for other important aspects of the motion (i.e., frequency content and duration). In order to overcome the deficiencies in peak ground motion parameters, parameters based on energy concepts and spectral response (as discussed below) have been developed (e.g., root-mean-square acceleration, power spectrum intensity, Arias intensity, response spectrum intensity). Descriptions of each of these parameters are found in Kramer (1996), Naeim (1989), and Naeim and Anderson (1993).

Frequency Characteristics of Ground Motions

To define the frequency content of the ground shaking, a response spectrum or Fourier amplitude spectrum is required. Of these, the response spectrum is most widely used in engineering practice, since it describes ground motion frequency characteristics in a form that is directly applicable to structural design and analysis. The ground response spectrum is obtained by applying the ground motions to the base of a suite of single-degree-of freedom oscillators all having equal damping ratios, and plotting the maximum response of the oscillator as a function of its natural frequency or natural period (Newmark and Hall, 1982; Kramer, 1996). This is depicted schematically in 1-6. This peak response can be plotted in either linear form using absolute acceleration and relative velocity and displacement, or in tripartite form using "pseudo"-accelerations, velocities and displacements (a simplified computational technique that relates each of the ground motion parameters by multiples of $2\pi/T$). These plots are particularly useful for demonstrating the predominant period of the earthquake motions.

Numerous empirical relationships for estimating response spectra for ground motions have been developed during recent statistical regression analysis of available strong motion data (Seismological Research Letters, 1997). The results of most of these recent studies have been presented in the form of simple, straightforward equations that allow the engineer to calculate spectral ordinates at the period of interest as a function of parameters such as earthquake magnitude, source to site distance, type of faulting, site geology, etc.

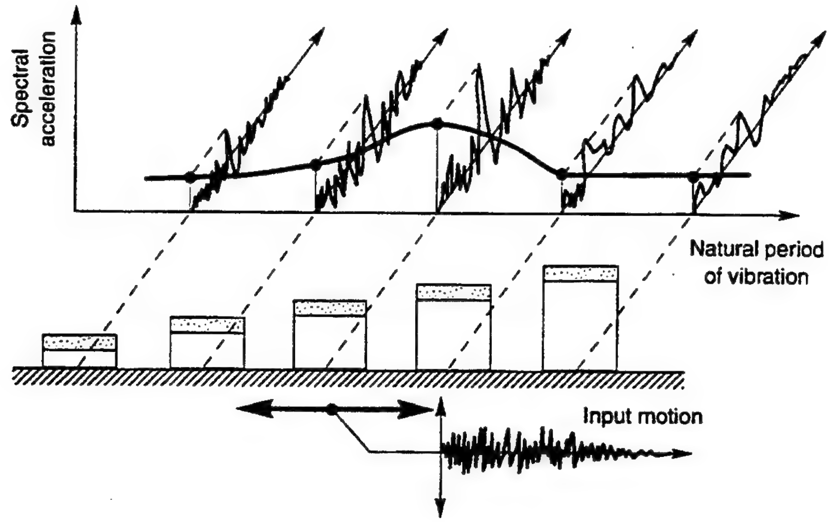


Figure 1-6: The Response Spectrum Obtained By Plotting The Spectral Accelerations Against The Periods Of Vibrations Of The S.D.O.F. Systems (Kramer, 1996)

As the seismic waves propagate from the rupture zone, the high frequency components of the motion are attenuated more quickly than lower frequency motions. This is due to damping by the transmitting rock which dissipates a fraction of the wave energy per cycle of travel. Since the high frequency waves have shorter wave lengths, they are attenuated more quickly with distance from the rupture than the longer period motions. The frequency dependent attenuation of ground motions results in a shift in the predominant period of the ground motions with increasing travel distance. The variation of predominant period at rock outcrops with magnitude and distance is shown in Figure 1-7.

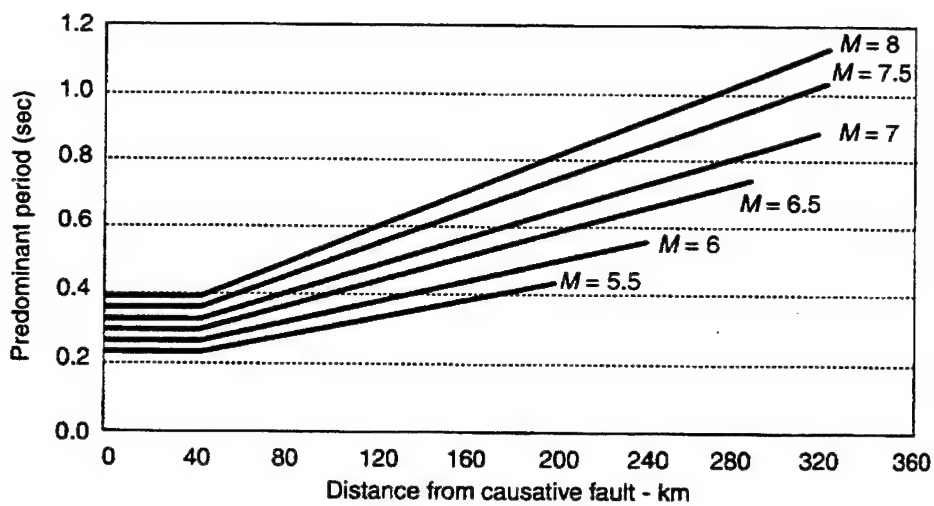


Figure 1-7: Variation Of Predominant Period At Rock Outcrops With Magnitude And Distance (After Seed Et Al., 1969)

Duration of Ground Motions

In addition to the strength and frequency content, the duration of the ground shaking will also influence the seismic performance of structures. This is particularly true for ductile structures designed to yield when subjected to strong ground motions. The inelastic response of such structures is sensitive to the number of cycles of strong motion that will be applied during the earthquake. The duration of shaking is also vital in the stability of cohesionless soils and performance of slopes and embankments.

The duration of strong shaking increases with increasing earthquake magnitude. The potential for earthquake-induced damage is a function of the duration of *significant* ground motions. For this reason, the concept of a "bracketed" duration has been used, which is defined as the length of time from the first exceedance of a specified acceleration level to the last exceedance of that acceleration level. Because the threshold for damaging motions is in the range of 0.05 g to 0.20 g for many structures, "bracketed" durations for acceleration levels of 0.05 g, 0.10 g and 0.20 g have been widely used (Naeim and Anderson, 1993). The variation of bracketed duration with magnitude and epicentral distance is shown in Figure 1-8.

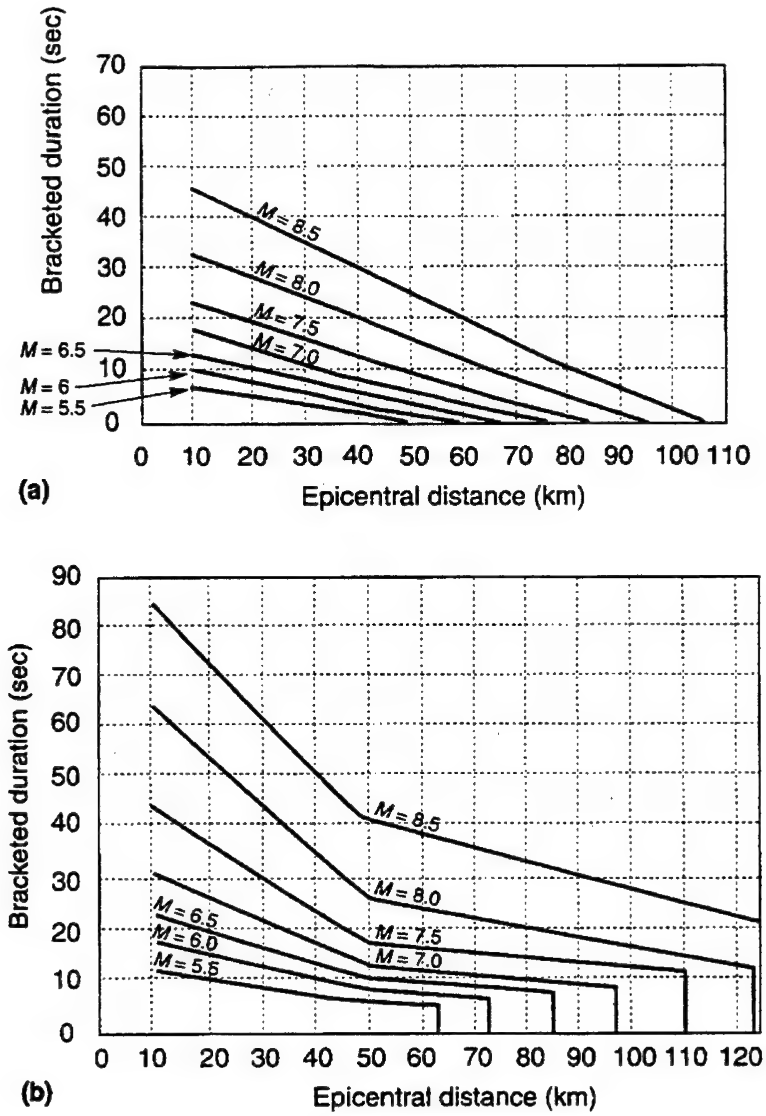


Figure 1-8: Variation Of Bracketed Duration (0.05 G Threshold) With Magnitude And Epicentral Distance: (A) Rock Sites; (B) Soil Sites (After Chang And Krinitzsky, 1977)

Factors Affecting Ground Shaking

The ground shaking characteristics at a particular site can be affected by numerous factors related to the fault rupture process, the propagation of the seismic waves as they travel from the ruptured fault to the site, and the local soil conditions at the site. These factors are briefly summarized below.

Fault Rupture Process The characteristics of the fault rupture that could influence the ground shaking at a site are the stress drop, the total fault displacement, the length of the fault break, the nature of the rupture process (i.e., the single or multiple fault breaks that can occur), the fault shape, and the proximity of the fault plane to the ground surface. In addition, whether the fault ruptures in a single direction or as bi-lateral rupture will significantly affect the duration of ground shaking. For example, both the 1989 Loma Prieta Earthquake ($M_w = 6.9$) and the 1995

Hyogoken Nanbu Earthquake ($M_w = 6.8$) featured bi-lateral rupture of the causative fault from the earthquake hypocenter. Because of this, the duration of the ground motions that were recorded during each of these earthquakes was much less than what might ordinarily be expected from earthquakes with the above magnitude levels. Although both earthquakes caused significant damage in the surrounding areas, these damage levels would undoubtedly have been much greater had the rupture been in a single direction rather than bi-lateral.

Travel Path Effects As seismic waves radiate away from the fault rupture zone during an earthquake the characteristics of the waves are modified. The strength of the ground shaking decreases due to geometric spreading of the wave front and damping of the waves as they propagate through the crustal rock. The frequency content of the motions is also affected by the dynamic behavior of the rock and the distance that the waves have traveled. The influence of the propagation path and transmission properties of the crustal rock on the seismic waves have been combined as "path effects." Once all potential seismic sources in the region of interest have been identified, the source-to-site distances can be scaled. Given the distance from the rupture to the site and a very general classification of the regional crustal rock, the path effects can be evaluated

As previously mentioned, geometric spreading and damping result in the attenuation of seismic waves. The decrease in the strength of the ground motions has been modeled numerically, although the most widely used attenuation relationships are based on statistical analyses of recorded ground motions. As regional arrays of strong motion instruments have become more common around the world, the data base of recorded motions is rapidly expanding. Statistical analyses of arrays of these recorded motions have been performed to develop a multitude of attenuation relationships for various regions, types of faulting, and site conditions. Recent overviews of this work are contained in the Seismological Research Letters (1997).

Many of the attenuation relations focus on the variation of peak acceleration or peak velocity with distance from the rupture zone. The example in Figure 1-9 shows the attenuation of peak acceleration and provides a comparison of several widely used empirical relationships for ground motions due to earthquakes in the western United States. Several factors must be considered when using empirical attenuation relationships:

- A variety of distance measures have been used in establishing the attenuation relationships. Such measures include; (a) distance to the rupture plane, (b) distance to the vertical projection of the rupture plane, (c) epicentral distance, and (d) hypocentral distance.

- The composition and integrity of crustal rocks will have a pronounced influence on the attenuation of ground motions. As illustrated in Figure 1-10, ground motions are felt over a much broader region in the eastern and central United States than in the western portion of the country. This is due to the age and composition of the rocks in the respective regions. In a general sense, the west coast is underlain by predominantly younger sedimentary rocks which have relatively high damping characteristics, while bedrock in the central and eastern regions is commonly much older metamorphic and igneous rock which has much lower damping and is much more efficient in transmitting the seismic waves. The empirical relationships are therefore only applicable to regions with roughly similar geology.

- The type of faulting and the depth of the rupture have been shown to influence the rate of attenuation. This is particularly evident with subduction zone earthquakes. As a result, in regions such as northernmost California and the Pacific Northwest that are prone to subduction zone earthquakes, attenuation relationships developed specifically for such earthquakes are used when evaluating seismic hazards associated with these thrust-type earthquakes (Seismological Research Letters, 1997).

- The near surface geology at the site is specified in several studies (hard rock, soft rock, shallow stiff soil, etc.).

For many years, very little strong motion data was available for source-to-site distances less than 15 km to 20 km. This was especially true for earthquake magnitudes in the range of engineering interest ($M \geq 6$). However, since the mid-1980s, analytical studies and analyses of near-field recorded motions have indicated that the position of the site relative to the fault could influence the characteristics of the ground motions at the site. The analytical studies demonstrated that, for a moving source, waves that leave the traveling rupture zone in opposite directions will have different amplitudes, much like the Doppler effect in acoustics. Near-field strong motion recordings (e.g., from the 1989 Loma Prieta, 1994 Northridge, and 1995 Hyogoken Nanbu earthquakes) have demonstrated the significance of near source effects such as rupture directivity, or "fling", for seismic design of structures located within about 10 km of the rupture zone.

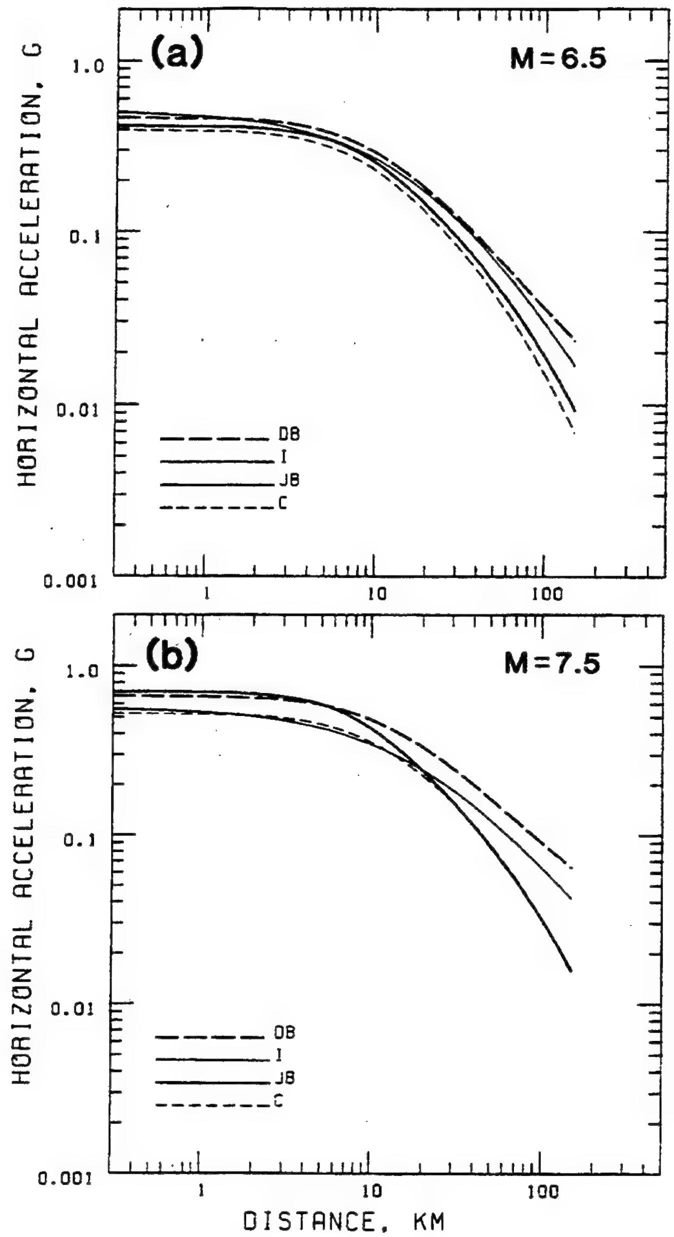


Figure 1-9: Comparison Of Different Relationships For Horizontal Acceleration At Magnitudes 6.5 (A) And 7.5 (B) (Joyner And Boore, 1988)

The propagation of the rupture toward a site at a velocity that is almost as large as the shear wave velocity of the rock causes most of the seismic energy from the rupture to arrive in a single large pulse of motion which occurs at the beginning of the record (Somerville et al., 1997). This large pulse results in enhanced near-field ground motions, particularly ground velocities and displacement components that effect longer period structures. The extent of this effect is quite variable, and depends on the azimuth of the site with respect to the direction of rupture. In addition, the strength and frequency content of the near-field ground motions can also be dramatically different, depending on the orientation of the measurement with respect to the source. Fault-normal and fault-parallel ground motions have been observed to be substantially different in the mid- to long period range (0.5 to 3.0 sec). The influence of these effects on the characteristics of strong ground motions should be incorporated into the design of mid- to long period structures as well as seismic base-isolation systems that are sensitive to large velocities and displacements.

Local Site Conditions The influence of local soil conditions on the strength of the ground shaking has long been recognized as a contributing factor to the geographic distribution of ground failures and structural damage during earthquakes. Of particular concern to earthquake engineers is the amplification of ground motions in period ranges of engineering interest, as well as the progressive softening (nonlinear behavior) of weak soils at high levels of shaking. The extensive collection of recorded strong motion records from earthquakes worldwide (e.g., 1985 Mexico City Earthquake, 1988 Armenian Earthquake, 1989 Loma Prieta Earthquake, 1994 Northridge Earthquake, 1995 Hyogoken Nanbu Earthquake) has contributed to an enhanced understanding of site effects for a wide variety of geologic conditions. These records have been used as the basis for quantitative studies of the influence of soil response on the characteristics of strong ground motions. The effects of site geology on the amplitude of various ground motion parameters such as peak acceleration, velocity and displacement, and also on the frequency content of the motions and their corresponding response spectra, have now been well demonstrated (Borcherdt, 1994; Seed et al., 1994). The potential for significant enhancement of ground motions in any period range is a function of seismological, geologic, and geotechnical factors, several of which are listed in Table 1-1.

In general, the results of seismicity evaluations are presented as the peak acceleration expected for a given return period or as a function of the probability of exceedance with time. In either case, the acceleration usually corresponds to the shaking at a rock outcrop, not at the surface of a soil profile. Site effects must then be evaluated as a function of key parameters such as; soil type, soil thickness, soil stiffness, and the strength of the bedrock

TABLE 1-1:FACTORS INFLUENCING THE MAGNITUDE OF SITE EFFECTS ON STRONG GROUND MOTIONS

SEISMOLOGICAL FACTORS
Intensity of bedrock, or input, shaking
Frequency characteristics of the input motions
Duration of the input motions
GEOLOGIC FACTORS
Soil type(s)
Thickness of the soil deposit
Underlying rock type
Geologic structure
GEOTECHNICAL FACTORS
Low-strain stiffness of the soils (shear wave velocity or maximum shear modulus)
Stiffness (impedance) contrast between the bedrock and overlying soils
Damping characteristics of the soil units
Cyclic modulus degradation behavior of the soils
Relationship between the shear strain and shear stress for predominant soil units
Site period
OTHER FACTORS
Two- and three-dimensional effects (e.g., subsurface bedrock topography, basin effects)

motions. Given this site-specific data, the dynamic response of the soils can be evaluated using either simplified empirical relationships or site-specific dynamic soil response techniques.

With this as background, the following subsections provide an overview of the various procedures available to estimate site-specific ground motions for the seismic design of port structures.

Estimation of Site-Specific Ground Motions

The estimation of site-specific ground motions for engineering design or analysis of port facilities is most typically based on: (a) deterministic or probabilistic methods for estimating site-specific rock motions (usually represented as peak ground acceleration or response spectra); and

(b) modification of these rock motions to account for local soil conditions. In addition, because of the increasing use of nonlinear methods of seismic analysis of port facilities (as discussed in following chapters), it is also often required to develop an appropriate ensemble of ground motion time histories.

Deterministic Methods for Estimating Site-Specific Rock Motions Deterministic methods for estimation of site specific rock motions consist of the following general steps: (a) for each of several potential seismic sources in the vicinity of the port site, estimate the maximum earthquake magnitude associated with that source; and (b) using an appropriate rock motion attenuation relationship, estimate the associated rock motions at the site, as a function of the maximum earthquake magnitude and the distance from the earthquake source to the site. After this is repeated for all potential earthquake sources in the vicinity of the site, the particular set of computed rock motions that lead to the most severe shaking at the site are selected. If a site has several different structures with different natural periods, response spectra that lead to governing motions in the period range of importance for each structure should be selected.

The above deterministic methods for estimation of site-specific rock motions have the advantage of being readily understood by non-technical port personnel and decision makers. However, they represent extreme earthquake scenarios only. Furthermore, they do not account for the uncertainties inherent in the estimation of the size and location of future earthquakes, and the rate at which rock motions attenuate with increasing distance from the seismic source. These factors are best represented using probabilistic methods summarized in the next section.

Probabilistic Methods for Estimating Site-Specific Rock Motions The seismic design or upgrading of a particular port component requires an assessment of the potential level of shaking at the site due to future earthquakes. In much the same way that port and coastal engineers design marine structures in consideration of the largest waves that may occur over the design life of the structure (e.g., 5-, 10-, or 25-year waves), earthquake engineers design for the levels of ground shaking that are anticipated to occur at a particular site with a specific average recurrence interval or return period (e.g., 72- and 475-years, which correspond to probabilities of exceedance of 50% and 10% respectively in 50 years).

This subsection provides an overview of probabilistic seismic hazard analysis (PSHA) methods for estimating site-specific rock motions with a given probability of exceedance or return period. The advantage of these methods is that they account for uncertainties in locations, magnitudes, and recurrence intervals of future earthquakes, and also in the rate of attenuation of rock motions with increasing distance from the seismic source. The seismic hazard analysis results are developed from information that describes the seismicity, geometry, and locations of the significant seismic sources in the region, and appropriate rock motion attenuation relationships for the region. Probabilistic models then synthesize this information to develop the probabilities or recurrence intervals associated with various levels of shaking at the site.

Figure 1-10 provides a flow chart that illustrates the PSHA procedure for developing site-specific uniform hazard spectra (which are spectra whose amplitudes at all natural periods represent the same probability of exceedance). PSHAs using this general approach have been utilized for numerous ports in the United States (e.g., Power et al., 1986; McGuire, 1990) and have formed the basis for the development of regional and national seismic hazard maps (Geomatrix, 1995; Hanson and Perkins, 1995; Leyendecker et al., 1995; Frankel et al., 1996).

However, in the implementation of these methods, the engineer should consider the following factors:

- A difficult element of the above probabilistic seismic hazard analysis process is the specification of the rate of seismicity (i.e., earthquake recurrence intervals) in a region. As previously mentioned, the relatively short historic record combined with varying rates of seismicity for the various tectonic provinces in the United States preclude a precise estimate for the recurrence intervals of damaging earthquakes. This uncertainty should be acknowledged and addressed in a straightforward manner. Probabilistic methods are commonly used to identify the uncertainties associated with seismicity rates

- The uncertainties in the rate of attenuation of the rock motions with increasing distance from the seismic source is represented in the PSHA either by using an appropriate probability distribution to represent this attenuation rate, or by performing a logic tree type analysis with mean values adjusted to reflect standard deviations in the empirical data.

Figure 1-11 shows the results of PSHA for soft rock-shallow stiff soil sites (NEHRP B-C boundary) in the United States. The parameter being mapped is the peak horizontal acceleration having a 10% probability of exceedance in 50 years. This corresponds to a roughly 475 year recurrence interval, and is equivalent to the exposure time for which the seismic load levels prescribed in current building codes are established. This data demonstrates relative ground shaking hazards for numerous regions in the United States. In light of the fact that seismic design at ports is most commonly based on exposure times of roughly 75 to 475 years,

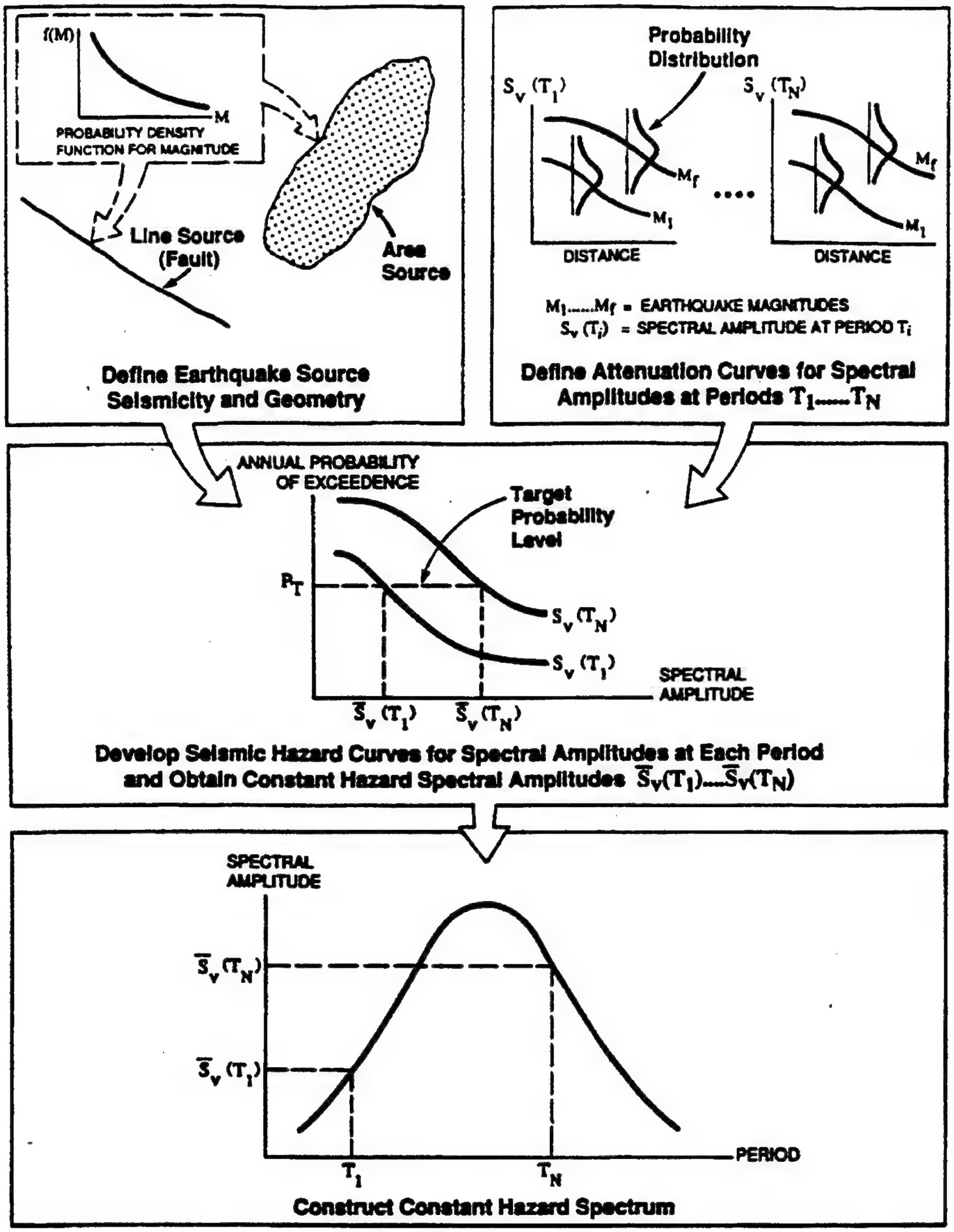


Figure 1-10: Development Of Uniform-Hazard Design Spectra Using Probabilistic Seismic Hazard Analysis Procedures (Werner, 1991)

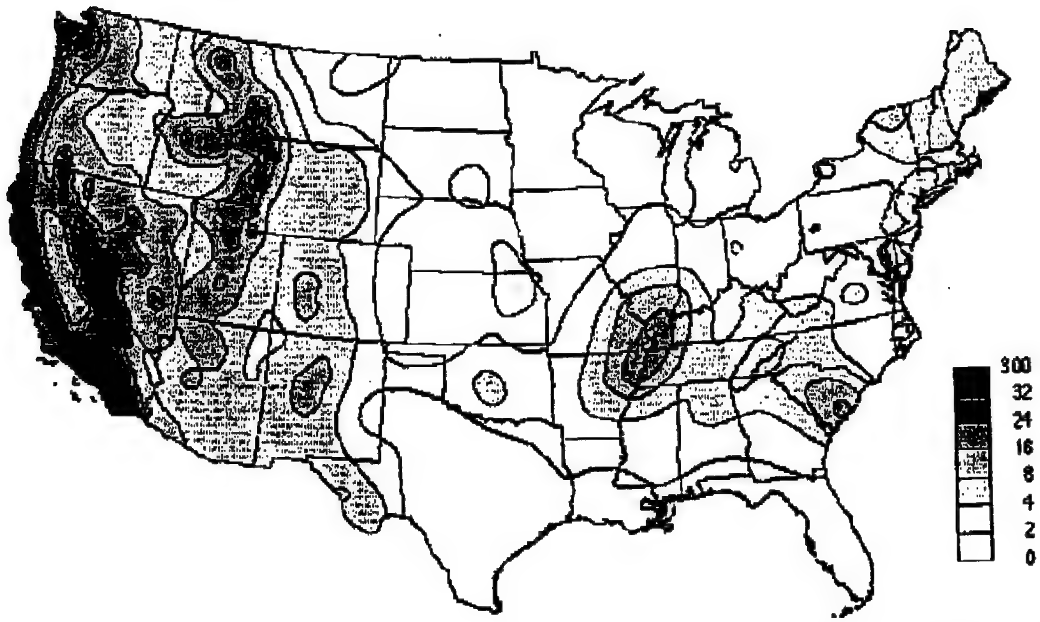


Figure 1-11: Peak Acceleration (%G) With 10% Probability Of Exceedance In 50 Years (Frankel Et Al., 1997)

Note that this map can be generated for all, or portions, of California.
The maps are available at the USGS seismic hazards and mapping web-site

it is evident that many regions of interest are considered prone to ground motions on rock that approach, or exceed, 0.10 g. Once the PSHA has been completed, maps such as this can be generated for a variety of ground motion parameters (e.g., peak ground motions, spectral accelerations at specified periods) and exposure times.

PSHA demonstrates the effect that the return period (or exposure time) has on the strength of ground motions anticipated at a specific site. The recurrence interval selected for the design of port facilities is a function of the seismic risk that can be accepted by the port authority. The variation in the peak ground acceleration having a 10% probability of exceedance is shown as a function of the exposure time in Figure 1-12. The data for this figure has been compiled from NEHRP (1993), Frankel et al. (1997) and Cox and Chock (1991). The results of probabilistic analyses such as these can be used by port engineers to assess the influence of exposure time on the seismic hazard.

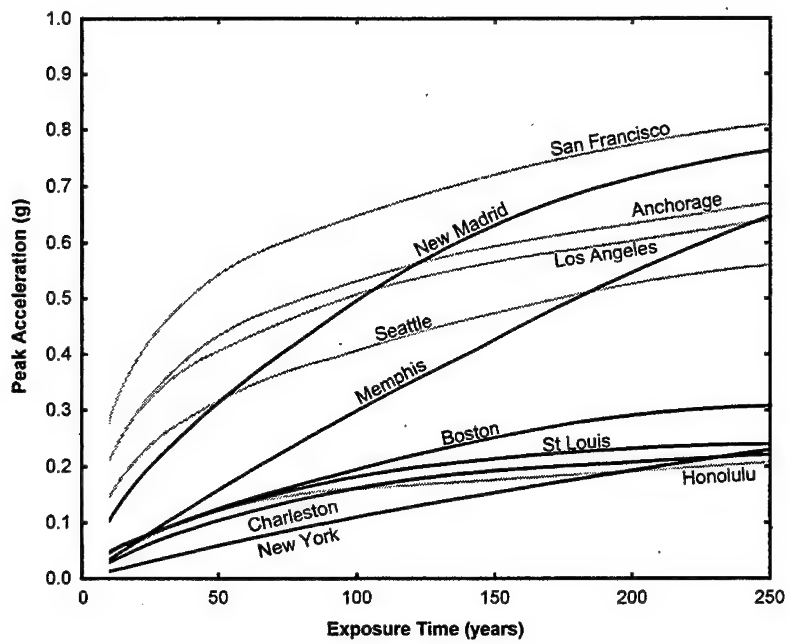


Figure 1-12: Peak Accelerations For U.S. Sites Assuming NEHRP B-C Rock Site Conditions (Motions Having A 10% Probability Of Exceedance During The Selected Exposure Time)

It is common for port engineers to use a performance-based criteria in seismic resistant design that requires the definition of two levels of ground motion for the design and analysis of structures. As an example, the guiding principles used for specifying the earthquake motions used in seismic design at the Port of Los Angeles (POLA) are as follows (Wittkop, 1997):

- Moderate earthquake motions (defined as Operating Level Earthquake motions, or Level 1 earthquake motions) should be resisted by wharf structures, retaining structure/dikes and critical operational structures and facilities founded on the backland fill areas, with only minor non-structural damage. From a design standpoint, deformations of wharf structures should not result in significant residual cracking or spalling of the concrete or permanent elongation of the steel reinforcement, and deformations of critical operational structures and facilities should remain within the elastic range. In their seismic design criteria, POLA defined the Operating Level Earthquake (OLE) motions as having a 50% probability of exceedance in 50 years (which is roughly a 72 year recurrence interval)

- Large earthquake motions (designated by POLA as Contingency Level Earthquake motions) should be resisted by wharf structures, retaining structure/dikes and critical operational structures and facilities in a manner which prevents collapse and major structural damage. Damage that does occur should be readily detectable and accessible for inspection and repair. Design concepts should be such that damage to foundation elements below ground level should be prevented. Container cranes and critical operational structures and facilities should remain operable with only minor repairs. The Contingency Level Earthquake (CLE, or Level 2) motions have been defined by POLA as having a 10% probability of exceedance in 50 years (or a 475 year return period). This is equivalent to the exposure time for ground motions used in the development of building codes.

It should be noted that this is just one example of performance-based seismic design criteria. Other ports may establish specific acceptable performance guidelines for different components based on the importance of the facility. Also, the exposure times selected by the Port of Los Angeles reflect the regional rate of seismicity. In the Los Angeles area, the Level 1 ground motions correspond to moderate levels of shaking that are likely to occur at least once during the life of the structure. Level 2 ground motions are much more severe levels of shaking that have a more remote potential for occurrence at the site during the life of the structure. For similar structures and construction practices, the exposure times adopted for use in seismic design will vary from region to region due to the variation in seismicity rates.

As noted previously, it has been common practice throughout much of the United States to use probabilities of exceedance of 50% in 50 years and 10% in 50 years to represent Level 1 and Level 2 ground motions for seismic design and analysis. However, this does not provide uniform protection across the state (e.g., Central Valley). To illustrate this across the United States, the data presented in Figure 1-12 has been replotted in Figure 1-13 in terms of the peak ground acceleration having a 10% probability of exceedance in a specified time interval divided by the peak ground acceleration having a 10% probability of exceedance in 50 years versus exposure time. These results show that for the western United States cities shown in Figure 1-13, the use of a probability level of 10% in 50 years to represent the Level 2 ground motions provides a reasonable estimation of the near-maximum levels of ground shaking that can occur over much longer exposure times. However, this is not true for sites in the central and eastern United States. For these regions, Figure 1-13 shows that the use of a probability level of 10% in 50 years may not provide adequate protection against the much larger near-maximum levels of ground shaking that can be associated with longer exposure times in these regions. The establishment of Level 2 ground motions for seismic design or retrofit of critical facilities at port in the central and eastern United States should carefully consider this trend, and may warrant the use of Level 2 ground motions with much longer exposure times for such ports. As discussed in other chapters, these considerations are now reflected in the new (1997) NEHRP seismic design provisions for buildings.

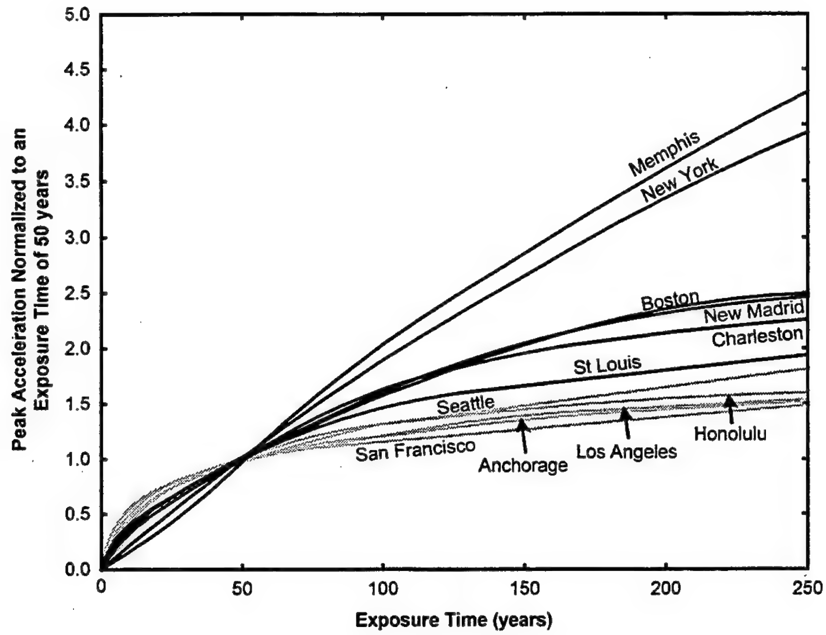


Figure 1-13: Normalized Peak Accelerations For U.S. Sites Assuming NEHRP B-C Rock Site Conditions

Local Soil Effects on Ground Surface Motions

Background Seismic waves which propagate from the underlying rock into near surface soil deposits are modified by the dynamic response characteristics of the local soils. The influence of the soil deposit on the bedrock motions will depend on the characteristics of the input motions, the thickness of the soil deposit, and the dynamic behavior of the individual soil layers. This aspect of the seismic hazard evaluation focuses on the dynamic response of soil deposits, or *site effects.*

In the last 15 years, the extensive collection of recorded strong motion records from worldwide earthquakes has contributed to an enhanced understanding of site effects for a wide variety of geologic conditions. The effects of site geology on the amplitude of ground motion parameters such as peak acceleration, velocity, and displacement, as well as the frequency content of the motions and their corresponding response spectra has been well demonstrated. An example from the U.S. Navy facility at Treasure Island during the 1989 Loma Prieta, Figure 1-14. The influence of the local soils on the characteristics of the ground motions is apparent. In addition to amplifying the peak ground acceleration, the dynamic response of the soil has resulted in enhanced motions at all periods between 0 and 4 seconds (as demonstrated by the response spectra).

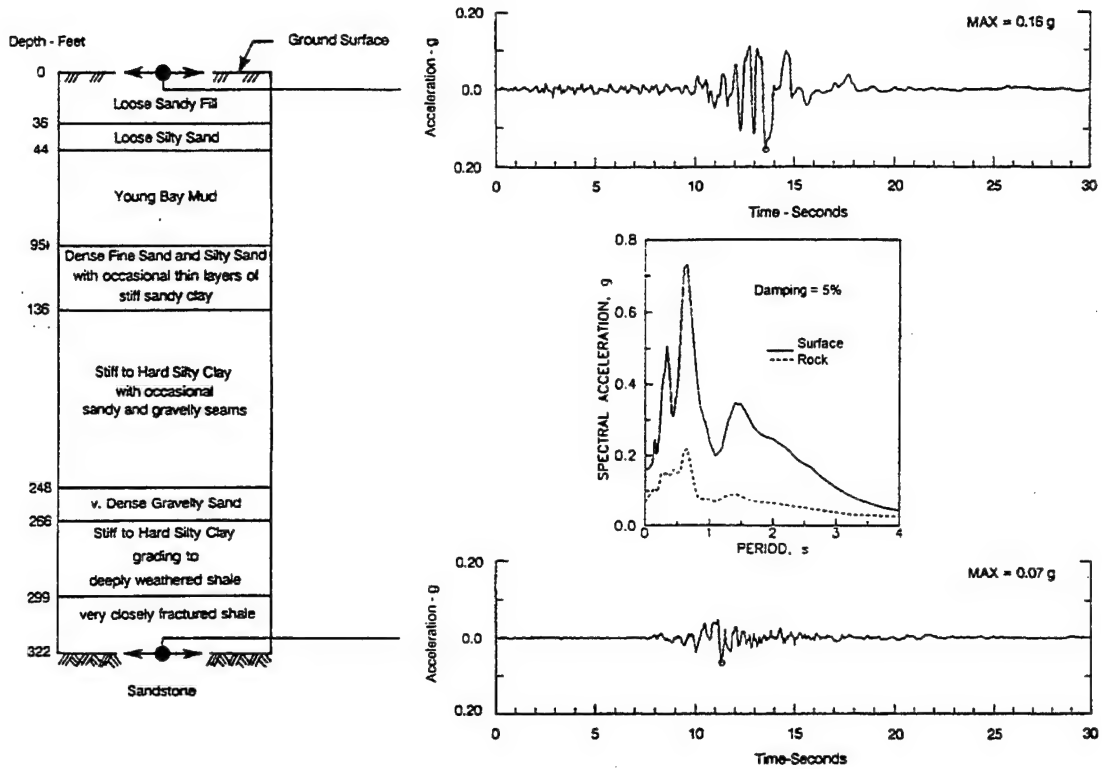


Figure 1-14: Soil Response At Treasure Island During The 1989 Loma Prieta Earthquake (After Seed Et Al., 1990)

Site effects can lead to enhanced ground motions at intermediate to longer periods of vibration, the range of concern for many structures. Spectral ratios (spectral acceleration of the ground surface motion divided by the spectral acceleration for the rock motion at the corresponding period) are commonly used to highlight the influence of the soil deposits on the characteristics of the strong ground motions. The spectral ratios for sites affected by the 1985 Mexico City Earthquake and the 1989 Loma Prieta Earthquake are shown in Figure 1-15. In both cases, the ground motions at intermediate periods (1 to 4 seconds) have been substantially amplified by the clayey soil deposits at the 15 sites documented. The relative amplification ratios are primarily functions of the stiffness of the clayey soils. The Mexico City clays are considerably less stiff than the San Francisco Bay muds.

Empirical studies of the effects of dynamic soil response on the characteristics of rock motions have been well documented in the geotechnical and seismological literature (e.g., Seed and Idriss, 1982; Borcherdt, 1994; Seed et al., 1994). These investigations have focused on two primary aspects of site response: (a) amplification of the peak acceleration on rock; and (b) amplification of spectral accelerations computed for the rock motions. Site soil effects on rock accelerations have been demonstrated in plots of PGA_{soil} versus PGA_{rock}, Figure 1-16. Given the peak acceleration for rock, the corresponding peak acceleration at the ground surface can be easily estimated. Similar plots have been developed for estimating spectral amplification ratios as well. In the aftermath of the 1989 Loma Prieta Earthquake, substantial research effort on this topic has led to the development of simple, yet suitably precise, techniques for developing

acceleration response spectra at soil sites. The methodology that has been adopted for use in current seismic design codes is presented below.

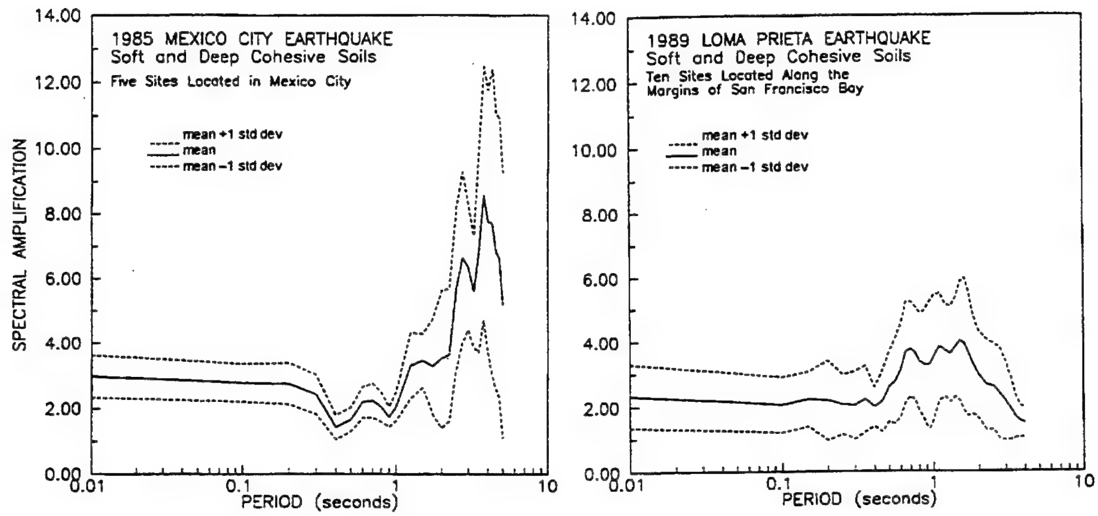


Figure 1-15: Spectral Amplification At Soft And Deep Cohesive Soil Sites (Dickenson And Seed, 1996)

Specification of Site Effects in Current Seismic Design Provisions The specifications that govern site effects in current building codes have been developed and adopted by a number of governmental agencies and engineering organizations over the past two decades. This summary focuses on the techniques for incorporating dynamic soil response in two current codes and recommended seismic design provisions (ICBO 1997; FEMA, 1998). This brief overview of the seismic design provisions is intended to highlight the strengths and limitations of these methods for use in practice.

The combination of the strong motion records obtained during the 1989 Loma Prieta Earthquake and extensive site characterization at strong motion instrument stations made possible with various in situ testing techniques (e.g., SPT, CPT, shear wave velocities) has provided the means for developing enhanced site classes for use in seismic design codes.

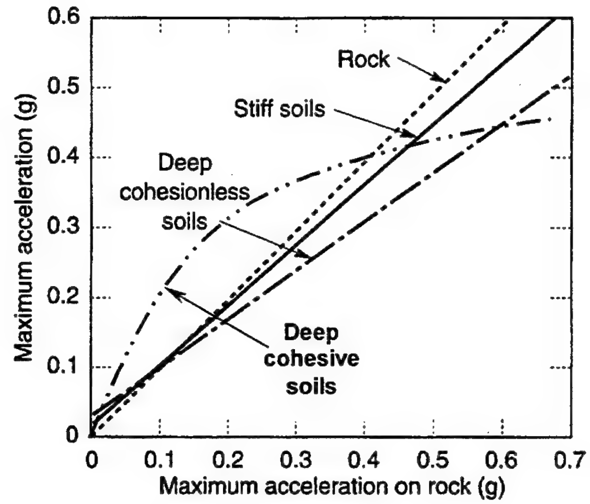


Figure 1-16: Approximate Relationships Between Peak Accelerations On Rock And Other Local Site Conditions (After Seed And Idriss, 1982; Idriss, 1990)

Comprehensive studies of recorded ground motions obtained for a wide variety of source characteristics and site conditions, and dynamic soil response analyses for a wide range of soil conditions have also been utilized to establish the expanded set of site classes that have been incorporated into current site coefficients and acceleration response spectra.

The new National Earthquake Hazard Reduction Program (NEHRP) and Uniform Building Code (UBC) provisions for site effects provide six well-defined site classes, as well as amplification factors that depend both on site conditions and on the level of the site-specific rock accelerations. The recommended site categories are specified in terms of the stiffness and strength of the upper 30 meters (100 feet) of the soil profile, Table 1-2. Exceptions to the 30 meters depth used for classification proposes are made for soil profiles that include very weak, metastable soils (site classes E and F). In these cases, thin near-surface layers can result in severe damage to foundations and retaining structures.

In addition to the incorporation of a more well defined site classification system, one of the primary improvements in the new seismic design provisions is the utilization of intensity-dependent amplification factors for modifying short- and intermediate period rock motions. The basis for the seismic hazard evaluation in current versions of the NEHRP and UBC seismic design provisions are spectral accelerations at selected response periods, and effective peak accelerations for rock sites, respectively. The respective ground motion parameters are obtained from maps then multiplied by site coefficients applicable to short-period motions and mid-period motions. These methods have purposely been developed so that it can easily be used with other site- or region-specific spectral maps which may be developed (e.g., Martin and Dobry, 1994; Geomatrix, 1995; Leyendecker, et al., 1995). The amplification factors, or *Site Coefficients*, provided in Tables 4-3 and 4-4 reflect both site effects at the different period ranges and the nonlinear behavior of soils. Given the site classification and values of A_a and A_v, the soil amplification factors can be determined.

TABLE 1-2:SITE CLASSIFICATIONS FOR USE IN SEISMIC DESIGN CRITERIA

SOIL PROFILE TYPE	GENERAL DESCRIPTION	SHEAR WAVE VELOCITY (m/sec)	STANDARD PENETRATION RESISTANCE (blows/30 cm)	UNDRAINED SHEAR STRENGTH (kPa)
A	Hard rock	> 1,524	n/a	n/a
B	Rock	762 to 1,524	n/a	n/a
C	Very dense soil and soft rock	366 to 762	> 50	> 96
D	Stiff soil	183 to 366	15 to 50	48 to 96
E	Soil profile with V_s < 183 m/sec, or any profile with more than 3 m of soft clay defined as soil with plasticity index > 20, water content > 40%, and undrained shear strength < 24 kPa.			
F	Soils requiring site-specific evaluations*.			

This straightforward technique provides a useful estimation of site specific soil response. The methodology involves the following steps;

a) Determine the "design-level" peak horizontal acceleration in bedrock (A_a) from: (a) available seismic zone maps; (b) an appropriate attenuation relationship; or (c) by means of site-specific seismicity studies.

b) Select a representative site category from Table 1-2. The site class is determined by obtaining an average shear wave velocity for the upper 30 meters of soil. The shear wave velocities are either measured using geophysical techniques, local shear wave velocity data in the same geologic units, or estimated from established correlations with other geotechnical properties (e.g., SPT or CPT penetration resistance, void ratio, undrained shear strength) for each of the foundation soils.

c) Select the short- and mid-period amplification factors (F_a, F_v) Table 1-3 and Table 1-4.

d) Compute spectral accelerations (S_A) at short periods using the formula $S_A = 2.5 \cdot F_a \cdot A_a$, and compute spectral accelerations at mid-to-long periods using the expression $S_A = F_v \cdot (A_a/T)$, where T is the period in seconds. Then, plot the elastic acceleration response spectrum (5% damping) as shown in Figure 1-17.

TABLE 1-3: VALUES OF Fa AS A FUNCTION OF SITE CONDITIONS AND SHAKING INTENSITY

SOIL PROFILE TYPE	SHAKING INTENSITY				
	$A_a < 0.1$	$A_a = 0.2$	$A_a = 0.3$	$A_a = 0.4$	$A_a > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	b
F	b	b	b	b	b

Notes:
a. Use straight line interpolation for intermediate values of A_a.
b. Site specific geotechnical investigation and dynamic site response analyses shall be performed.

TABLE 1-4. VALUES OF Fv AS A FUNCTION OF SITE CONDITIONS AND SHAKING INTENSITY

SOIL PROFILE TYPE	SHAKING INTENSITY				
	$A_v < 0.1$	$A_v = 0.2$	$A_v = 0.3$	$A_v = 0.4$	$A_v > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	b
F	b	b	b	b	b

Notes:
a. Use straight line interpolation for intermediate values of A_v.
b. Site specific geotechnical investigation and dynamic site response analyses shall be performed.

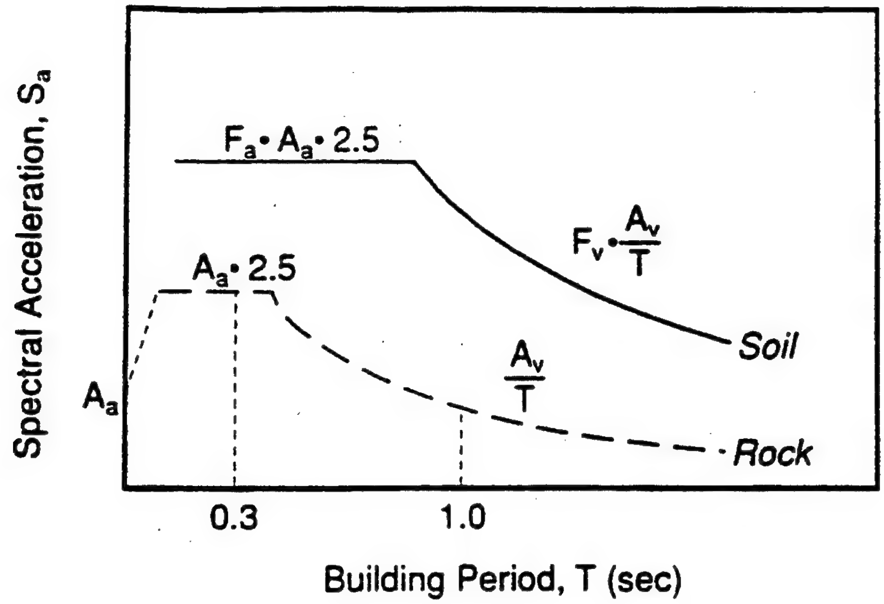


Figure 1-17: Two-Factor Approach To Defining Design Spectra (FEMA, 1995)

Numerical Ground Response Analysis Methods for Evaluating Site Effects

Despite recent improvements in the methods for constructing soil-dependent acceleration response spectra now contained in the NEHRP and UBC seismic design provisions, aspects of the soil profile or structure under consideration may warrant a site-specific response analysis. In certain instances, the seismic design provisions in building codes prescribe that the simplified methods of evaluating dynamic soil response should be augmented with the results of site-specific response analyses. Seismic design which accounts for near-source effects, soft or potentially unstable soils, critical structures, or structures with plan-irregularities, may require more rigorous response analyses than are outlined in building codes.

The engineer has at his or her disposal a variety of computer programs that can be used to predict the dynamic response of soil deposits. The level of sophistication of these numerical methods (and the soil data and engineering time required) varies considerably with the more complex programs requiring as many as 20 soil parameters for each soil layer in the model. In addition, the computer programs that have been developed for modeling dynamic soil response rely on various simplifications and assumptions in order to solve equations for wave propagation through soils. The spectrum of computer-based analyses for dynamic soil response ranges from relatively simple linear-elastic total stress soil models to sophisticated and fully nonlinear effective stress techniques. A cursory introduction to dynamic soil models is provided, followed by several practical insights on the performance of analytical soil response programs.

The influence of the soil deposit on the amplitude of the incident seismic waves will be greatest near the predominant period of the deposit. This period can be estimated for the case of vertically propagating waves in a linear elastic soil media from the relation:

$$T = \frac{4H}{(V_s)_{AVG}}$$

where H is the depth of the deposit and $(V_s)_{AVG}$ is the average shear wave velocity of the deposit. While this simple relationship provides a useful insight into the period range at which site effects may be significant, it does not address the magnitude of this amplification on the ground motions. This amplification is a function of the thickness and stiffness of the soils, the contrast between the stiffness of the soil and underlying rock (*impedance contrast*), and the strain-dependent properties of the soil. The response of a multi-layered soil profile subjected to transient motions is a complex phenomenon which involves strain- and frequency-dependent behavior, hysteretic stress versus strain soil properties, and potential fatigue related phenomena such as modulus degradation and excess pore pressure generation. This behavior is clearly nonlinear and difficult to model analytically. In order to account for these and other factors in the analysis of dynamic soil response, computer programs must be employed.

Requisite input parameters and modeling details for these analyses of seismic soil response include: (a) suitable strong motion records (digitized acceleration time histories); and (b) representative dynamic properties for soils at the site. In addition to the unit weight (γ_t), which can be readily estimated, the two principal dynamic soil properties of interest in response analyses are: (a) the dynamic shear modulus, G, which describes the stiffness of the soil; and (b) some measure of dynamic material damping (i.e., the damping ratio, β, which is related to the energy lost per cycle of shaking).

The shear wave velocity is a useful parameter for describing the small-strain (cyclic shear strains $\leq 1.0 \times 10^{-4}\%$), and the corresponding maximum stiffness of the soil. The shear wave velocity can be measured in situ using common testing techniques, and it is now used as one of the criteria for classifying soil deposits in seismic design codes, Table 1-2. The shear wave velocity is related to the dynamic shear modulus of the soil by the simple formula:

$$G_{\max} = \frac{V_s^2 \gamma_t}{g}$$

where G_{max} is the small strain dynamic shear modulus, V_s the shear wave velocity, γ_t the total unit weight of the soil, and g is the acceleration of gravity. Despite recent advances in sampling and laboratory testing techniques, the adverse effects of unavoidable sample disturbance on the small strain dynamic modulus of a soil, as well as difficulties associated with small-strain measurements, render in-situ seismic wave velocity measurements the currently preferred method for determining G_{max}.

Both dynamic shear modulus and damping are "nonlinear" properties of soils: both are strongly dependent on shear strain levels. As shear strains increase the dynamic moduli decrease and damping increases as shown in Figure 1-18. Computer programs that are most commonly used in engineering practice for the development of a site-specific acceleration response spectrum are based on the assumption of vertically propagating seismic waves through horizontally layered soil deposits. These simplifications allow the response analysis to be

performed on the basis of one-dimensional (1-D) wave propagation. For most engineering work, the assumption of vertically propagating waves is not unreasonable, due to the refraction of waves at layer interfaces as the waves travel from deep, dense material upward through soils which are progressively less-dense and subjected to reduced confining stresses. The limitations imposed by 1-D analyses include several effects that can influence site-specific ground motions. Among these effects are two-dimensional and three-dimensional (2-D and 3-D) bedrock topography, basin effects, wave-scattering, horizontally propagating surface waves, and sloping ground conditions.

Equivalent Linear Dynamic Soil Response Method The most commonly used equivalent linear soil response model is incorporated in the program SHAKE which was originally developed by Schnabel et al. (1972) and later updated by numerous individuals (e.g., Idriss and Sun, 1992). The SHAKE program employs an equivalent linear total stress analysis to compute the response of a horizontally layered visco-elastic system subjected to vertically propagating shear waves. In this, an exact continuum solution ("shear-beam") to the wave equation is adapted for use with transient motions, through the Fast Fourier Transform (FFT) algorithm. The FFT essentially replaces the transient motion represented by the digitized acceleration time history by a finite series of harmonic motions. The hysteretic stress-strain behavior of soils under symmetrical cyclic loading is represented by an equivalent modulus, G, corresponding to the secant modulus through the end points of the hysteresis loop and an equivalent damping ratio, β, corresponding to the equivalent damping. The equivalent modulus and damping ratio are equivalent-linear, strain-dependent properties. Modulus reduction and damping curves (such as those shown in Figure 1-18) are incorporated into input files to model the nonlinear dynamic properties of the soils.

The shear moduli and damping corresponding to the computed shear strains are determined using an iterative procedure that is based on linear dynamic analysis. For all soil sublayers estimates of the dynamic moduli (G_{max} or V_s) and a damping ratio ($\beta \approx 1$ - 5%)

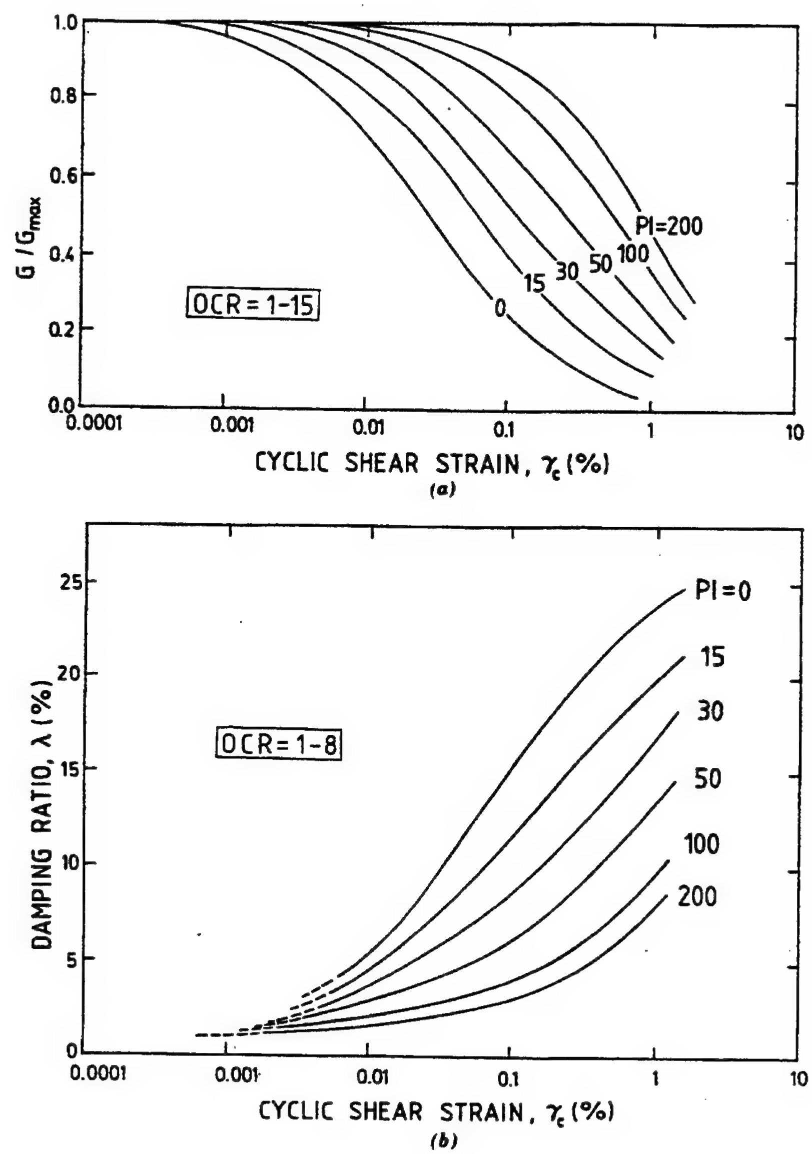


Figure 1-18: Relationships Between The Shear Strain And The Shear Modulus And Damping Ratio For Normally And Overconsolidated Soils (Vucetic And Dobry, 1991)

are provided for the first iteration. The equivalent linear method incorporated in SHAKE then approximates nonlinear soil behavior with an iterative method that uses a linear wave propagation formulation with soil properties that are compatible with equivalent uniform, or "effective", shear strain levels which are assumed to exist within each of the soil sublayers for the duration of the excitation. The ratio of this equivalent uniform shear strain to the calculated maximum strain is specified as an input parameter (**n**) and the same value of this ratio is used for all sublayers. The ratio **n** is computed using the simple formula proposed by Idriss (Idriss and Sun, 1992); **n** = (**M**-1)/10 where **M** is the moment magnitude of the earthquake being modeled. At each iteration, **n**% of the peak strains computed at the mid-point of each soil sublayer from the previous iteration are used to obtain new values of strain-dependent modulus and damping ratio. The program iterates until the modeled strain-dependent soil properties are compatible with

the strain levels associated with the calculated response of the system. The final computed soil properties are thus referred to as "strain-compatible" properties

The widespread use of SHAKE, and other similar equivalent linear methods in engineering practice is due to its relative ease of use and the limited number of input parameters required for each soil layer. In addition to the digitized acceleration time history and the strain-dependent modulus and damping curves, each soil layer is completely described its thickness, unit weight, low-strain modulus and damping. The simplicity of the SHAKE model results in an economy of effort when preparing input files and interpreting computer-generated output.

Although the equivalent linear method of analysis has been found to provide acceptable results for many engineering applications, a number of limitations have been noted in the technical literature. In addition to the inherent limitations of 1-D response analyses previously noted, practicing engineers should be aware of the following issues when performing equivalent linear dynamic response analyses:

- The equivalent linear model is based on a linear elastic formulation. Because all hysteresis loops are symmetric about the origin, permanent (plastic) deformations are not modeled.

- Once the strain-compatible soil properties have been obtained for each layer, single values of G and β are used throughout the final analysis. Moduli and damping ratios are independent of frequency. In addition, the soil properties associated with the largest strains are slightly "under-softened" and under-damped, while those at lower strains are over-softened and over-damped.

- The equivalent linear method employed in SHAKE performs a total stress analysis. Although the output (uniform stresses or strains) can be used in a decoupled analysis of excess pore pressure generation, the program does not perform an analysis wherein the stiffness of the soil is modified at each iteration to account for generated excess pore pressures. The excess pore pressures that can be generated in loose to medium dense sands and sensitive cohesive soils are not accounted for. These excess pore pressures result in progressive softening of the soil, a reduction in the high frequency components of the motions, and potentially large permanent displacements.

- No upper limit is placed on the peak equivalent uniform shear stress that is computed in each layer. In cases where moderate to strong levels of shaking are input at the base of a soil profile which includes soft to medium stiff cohesive soils the computed shear stresses often exceed the dynamic shear strength of the soil. The result is overprediction of peak ground accelerations and high frequency motions. The strain dependent soil properties can be modified to reduce the computed stresses.

Despite these limitations, numerous validation studies have demonstrated the accuracy of the 1-D, equivalent linear method to model the dynamic response of various soil profiles for which ground surface and representative rock input motions are available (Seed et al., 1994; Idriss, 1993a).

Fully Nonlinear Dynamic Soil Response Methods In order to overcome the deficiencies of total stress equivalent linear soil response methods, fully nonlinear effective stress analyses have been formulated. These numerical methods are based on a time-domain solution wherein the response of the soil is evaluated in a stepwise manner at each point in the acceleration time history. The fully nonlinear methods offer a number of improvements in the computation of dynamic soil response in that the following effects can be accounted for: (a) strain- and frequency-dependent soil properties; (b) more accurate modeling of the stress-strain response of the soil; (c) specification of peak undrained shear strengths; and (d) the generation and dissipation of excess pore pressures can be included in the analysis.

In order to model the dynamic behavior of the soil during loading, unloading, and reloading, additional soil parameters are required for each layer. From a practical standpoint, the application of these models is often precluded by the cost of obtaining representative soil properties required as input information. This occasionally leads to the reliance on default values that may be supported by relatively few laboratory investigations or well-documented case histories.

Among the more commonly used 1-D nonlinear programs are DESRA-2C (Lee and Finn, 1991) and SUMDES (Li et al., 1992). Although a complete description of the constitutive models incorporated into these respective programs is beyond the scope of this summary, both of these programs are capable of performing total stress, as well as coupled effective stress analyses. The constitutive relationships utilized in DESRA-2C for an effective stress dynamic response analysis with redistribution and dissipation of porewater pressure require 18 material constants for each soil layer. The hyperbolic stress-strain relationship is used and these soil parameters describe the unit weight, maximum shear strength, maximum shear modulus, stress-strain behavior, volumetric strains, material hardening, permeability, and viscous damping. The computer program SUMDES incorporates a sophisticated plasticity model (termed the *bounding surface hypoplasticity model*) that is based on critical state soil mechanics. The program uses a multi-directional formulation and plasticity models for soil behavior that facilitate the modeling of shear waves and compression waves simultaneously. With this technique, horizontal motions, vertical settlement, shaking induced lateral stress variations, soil compression and dilation, liquefaction behavior of sandy soils, and rotational shear can be modeled. Five levels of analysis are available with the two most complex models requiring 19 to 20 soil parameters for each layer.

The dynamic modeling capabilities of these programs clearly exceeds that provided by the equivalent linear methods. Potentially important dynamic soil behavior such as progressive softening due to the generation of excess pore pressures, limiting shear strength and permanent soil deformations can be evaluated with these fully nonlinear soil models. However, in practice, the advantages provided by the fully nonlinear soil response programs must be weighed against the cost of laboratory testing programs required to obtain representative soil properties for the analyses, as well as the engineering time necessary for the development of input files, performance of parametric studies, and additional scrutiny of the analytical results. The engineer must balance economy of use with usefulness of the output. For example, a response analysis of a shallow, stiff clay deposit under moderate levels of shaking will likely not warrant a fully nonlinear effective stress analysis. Conversely, the characterization of the dynamic response of an extensive deposit of medium dense saturated sandy soil subjected to similar levels of shaking

may require a coupled effective stress analysis. In either case the results of these analyses should always be tempered with sound engineering judgment.

Ground Motion Time Histories

Requisite input for numerical seismic analyses of soil deposits and/or port facilities include time-history representations of the site-specific ground motions. Digitized accelerograms (i.e. acceleration time histories) are the most common form of seismic input employed in numerical models. These accelerograms should be consistent with the design spectra developed for the site, and should represent the anticipates shaking at the site due to all of the significant potential earthquake sources in the vicinity of the site. The development of ground motion time histories for numerical analyses, as well as for general seismic design applications, has been based primarily on measured and processed accelerograms contained in the current strong motion database. In addition, various types of synthetic accelerograms have been used for certain applications.

Strong Motion Records A key source of motion-time histories for seismic design purposes is the strong motion data base, which contains an extensive array of recorded and processed accelerograms. Agencies such as the U.S. Geological Survey, NOAA National Geophysical Data Center, and the California Division of Mines and Geology distribute digitized strong motion records that can be used for seismic analyses and design. The various stations at which these accelerograms were recorded represents a wide variety of geologic, tectonic, and subsurface soil conditions -- all of which can influence the characteristics of the recorded motions. Differences in the conditions at the instrument locations for the various accelerograms are undoubtedly the source of the marked differences in the features of these recorded motions. Therefore, the engineer must be aware of the significance of these conditions, and should judiciously select an ensemble of accelerograms that collectively best represent the particular conditions at the project site. In addition, such accelerograms may be adjusted, where appropriate, such that; (a) the composite spectra from the accelerograms are reasonably consistent with the design spectra developed for the structures; and (b) the composite characteristics of the accelerograms are reasonably consistent with those indicated by applicable ground motion attenuation relationships. In many regions of the United States, the lack of a robust collection of strong motion recordings precludes the acquisition of a representative ensemble of natural time histories for seismic design purposes. In addition, site-specific aspects of a particular project (i.e., location relative to the seismic source, local geography and geologic setting) may eliminate the existing record from consideration even in well-instrumented, seismically active portions of the country. In cases such as these, synthetic earthquake ground motions can be generated.

Synthetic Accelerograms The development of synthetic accelerograms was first motivated by the need to partially fill important gaps in the current strong motion database. Toward this end, various methods based on random vibration theory or on analytical wave propagation models were used to develop synthetic accelerograms that embody the basic characteristics of strong motion records, as indicated by available data and by engineering judgement regarding future earthquakes (Silva and Lee, 1987).

In practice, it is common for the design ground motions to be described in terms of a peak acceleration and a specified target spectra or design response spectra. The design response spectra may be established using empirical equations as described above. The spectral shapes obtained using these equations are generally smooth and somewhat "broad band" in that they do not exhibit the peaks and valleys characteristic of the response spectra computed from natural ground motions. The corresponding synthetic motions are then produced so that their response spectra closely matches the smooth design response spectra.

It should be noted that synthetic ground motions developed in this manner are often more robust than actual ground motions, due to the fact that they are constructed to match a broad, smooth spectrum and therefore contain significant seismic energy at all frequency contents. As a result, such synthetic accelerograms may not be appropriate for use in analyzing the nonlinear response of structures or soil deposits, which will be strongly dependent on the signature of the input motions. Therefore, care and judgment should be exercised when identifying suitable spectrum-compatible accelerograms for use in nonlinear structural analyses, and when interpreting the results of such analyses.

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CHAPTER 2 COMPONENT DESIGN AND EVALUATION CONSIDERATIONS

Introduction

Marine oil terminal components are quite broad and varied, and include a range of earthen embankments and berthing structures. The earthen embankments may be plain, armored with rock rip rap or other materials, and may possibly be topped with a concrete structure. Berthing structures at ports may be massive concrete block gravity structures, steel sheet-pile retained earth structures, pile supported marginal wharves, pile-supported piers, or combinations of these.

This chapter provides guidelines to assist the engineer in addressing earthquake engineering aspects of these components. It is organized into subsections which cover the most common waterfront components at ports. This material provides seismic guidelines for specific types of port waterfront components – examples include embankments, piles (which are a common element for many types of waterfront and other types of port components), marginal wharves, gravity retaining structures, and steel sheet-pile wharves. For each of these components, these sections summarize general functional/operational requirements, guidelines for establishing seismic performance requirements, performing preliminary seismic evaluations, and for seismic analysis, seismic design of new components, and seismic retrofit of existing components.

General Seismic Performance Issues for Waterfront Structures

As outlined in the seismic criteria a two-level design approach for port structures has been widely adopted. The Level 1 design considers a moderate level of ground shaking that is likely to occur during the life of the structure (also often termed the Operating Level Earthquake (OLE) ground motions). Under this level of ground shaking, the structure is designed so that its operations are not interrupted and any damage that occurs will be readily repairable within a relatively short time. The Level 2 design considers much stronger motions that are less likely to be exceeded during the life of the structure (commonly called the Contingency Level Earthquake (CLE) ground motions). Under these motions, the structure is designed so that any damage that occurs is controlled and repairable (although possibly over an extended time). In this, the Level 1 design criteria address economic issues associated with a loss of operations at the port and major repair costs, and the Level 2 design criteria address the same issues, and the additional considerations of life safety, structure repairability, environment protection, and collapse avoidance.

Recent guidelines indicate that typical building codes are usually not appropriate for port waterfront components, and recommends that such components be designed in accordance with the specific seismic performance requirements for the component as well

as its physical attributes (Werner, 1998). By including other design references, it is recognized that this may develop a dual design criteria because of local building code authority jurisdiction over some waterfront structures. These seismic performance requirements (and associated design criteria) should not only address life safety, but should also consider the importance of the component to overall port operations as well as any special requirements of the component (such as special pollution control requirements), etc. Likewise a reassessment may be required after the occurrence of an earthquake.

The development of more stringent seismic performance requirements and design criteria for waterfront components is on an upswing in the United States. This appears to be due to an increased awareness of the lessons learned from past earthquakes regarding the extent and consequences of inadequately designed waterfront components, and measures that can be implemented to improve the seismic performance of these components.

Embankments

Embankment Types

Earth embankments are commonly the most prevalent waterfront components in ports due to their widespread use as perimeter containment dikes during initial reclamation operations and as breakwaters which protect the inner port from wave action and current-induced scour. This section identifies the most common embankment types and focuses on the seismic performance criteria that are common to each.

Native Soils Natural soil deposits that form banks such as spits or levees can be loosely categorized as embankments, since these natural barriers can provide protection for harbors or river front ports, and these soils are commonly incorporated into engineered earth structures. It is common for engineered fills to be placed on existing rises of native soil in order to minimize the volume of soil required during construction of earth embankments. The heterogeneous and usually weak nature of these native deposits can result in embankments that are marginally stable during an earthquake, and are also prone to loss of strength due to groundwater seepage conditions during extreme high tides at coastal ports or during flood stages along inland waterways. The seismic performance of embankments made of, or on, these deposits has generally been very poor.

Rock and Sand Dike with Backland Fills Rock and sand embankments have been used extensively as perimeter dikes during the construction of offshore reclamation projects. The costs associated with the excavation and transport of the materials will usually determine the relative volumes of sand or rock used in construction of the embankment. In many regions, the inherent benefits of using rock fill in construction are overshadowed by the relatively high cost of transporting the material from distant quarries. A multitude of different embankment configurations have been employed at ports to optimize the use of soil and rock fill. Examples of these embankment types include single lift sand dikes

covered with rock armor for wave protection, single lift rock fill dikes, multiple lift rock fill dikes and hybrid dike configurations (Figure 2-1).

The addition of soil fill behind the dikes creates the backland areas of the port. This fill can be placed either concurrently with the construction of the dikes or after construction of the entire dike has been completed. It is quite common for hydraulic placement methods to be used for the fill behind the dikes. In several instances (e.g., at the Port of Osaka, Japan) fine-grained, bay floor sediments have been used as backland fill. This practice has led to long-term settlement problems associated with consolidation of the fill soil. In most cases however, sandy soils are used as backland fill. When placed through slurry pipes or end-dumped through standing water from barges, these sandy soils are very loose and prone to earthquake-induced liquefaction.

Breakwaters Various types of breakwaters used for offshore wave protection at ports are shown in Figure 2-2. The most common types are rubble-mound sloping-type breakwaters (Figure 2-2a), composite-type breakwaters (Figure 2-2b), and, to a lesser degree, specialized breakwaters such as curtain walls, sheet-pile breakwaters, and floating breakwaters. This discussion focuses on rubble-mound and composite-type breakwaters due to their common usage and similar foundation requirements.

Rubble-mound breakwaters, Figure 2-2a, are constructed in much the same manner as sand- and rock-dikes. Additional rock layers are placed on the breakwaters in order to provide protection from the combined action of direct wave impact and littoral currents. These layers are often augmented with shape-designed concrete blocks for the dissipation of wave energy. Composite-type breakwaters, Figure 2-2b, differ from rubble-mound breakwaters in that a soil and rock berm serves as the foundation for a gravity wall (usually a concrete caisson) which acts as the breakwater. The foundation requirements for these two types of breakwaters are similar, although the rubble-mound-type are commonly wider at the base due to the slope angles used in construction.

Bulkheads and Sea Walls Bulkheads and sea walls are onshore structures that serve as both earth retaining systems and wave protection structures. These waterfront structures include gravity walls, cellular sheet-pile bulkheads, anchored sheet-pile walls, and composite concrete faced walls. Seismic guidelines for these structures are provided in subsequent sections.

Bulkheads are usually vertical in section to facilitate berthing for ships, while sea walls are commonly tiered or sculpted to optimize the dissipation or redirection of wave energy. The techniques used for backfilling these structures are equivalent to the hydraulic methods used for reclamation behind dikes. For this reason, the seismic performance of these structures has been similar to that of the other embankment types.

General Functional/Operational Requirements at Ports

As categorized in this subsection, embankments are earth structures, or composite structures that function as either earth-retaining systems, shore-protection components, or both. Functional/operational requirements of the various embankment types are summarized below.

Sand and Rock Dikes with Backland Fills. These dikes are used as perimeter-retaining structures around reclaimed land such as islands and marginal wharves, and also as foundation pads for gravity structures such as caissons and embankments for ground transportation systems. They form the interface between the marine and backland portions of the port. Although these structures are used for earth retention, they are quite distinct from other structural earth-retaining systems which are discussed in subsequent on gravity-retaining structures and steel sheet-pile retaining structures.

Breakwaters. Breakwaters protect the harbor and shore areas from the waves and currents generated at sea. This provides for calm water on the leeward side of the breakwater within the harbor and reduces navigation hazards. In addition, breakwaters can be used to mitigate the migration of sediments into the harbor. Breakwaters are most commonly gravity structures and they can be isolated offshore or connected to land. Both the offshore and land-connected breakwaters are sometimes used as docks.

Sea Walls and Bulkheads. Sea walls are also used as wave protection, although these walls are located along the shoreline and protect the shore from erosion due to wave action and littoral currents. Bulkheads are waterfront retaining walls that include gravity-type quay walls and sheet-pile structures. These structures form the marginal wharves and piers along which berthing and cargo handling operations take place. Their primary function is to maintain adequate freeboard to preclude overtopping by waves. This function is impaired if the structure settles or topples during an earthquake. These potential failure modes are distinct in that settlement is due to densification or deformations of the foundations soils, whereas toppling or sliding may be due to inadequate dimensioning of the embankment.

As the primary waterfront components at most ports, earth retention embankments (dikes and bulkheads) often provide foundation support for pile-supported structures, utility lines, and cargo handling components , for loading and unloading of ships, such as cranes, ramps, and conveyance systems. Embankment failures are manifested as excessive lateral and vertical deformations. These deformations will, in turn, result in damage to the port components located near the waterfront and the disruption of port operations. Given the importance of these components to port operations, the primary requirement of the embankments is that ground deformations be minimized. These include deformations of the foundation soils as well as the embankments themselves. Localized failures such as slumping of the face of the embankment or sliding of armor layers could affect embedded piles or expose the earth structure to wave induced scour. The latter effect would be relatively easy to remedy and would be considered an acceptable consequence of a design-level earthquake in most cases.

Guidelines for Developing Seismic Performance Requirements

The weak cohesive soils and potentially liquefiable sandy soils that are common throughout the marine environment are primary factors in most embankment failures. This observation has been made for failures due to static and dynamic loading conditions. The seismic performance requirements for embankments should reflect the sensitivity of adjacent port components to ground deformations. For example, acceptable deformation limits for sand and rock dikes will vary, depending on whether the earth structure: (a) is placed as a perimeter dike adjacent to a storage yard or a relatively undeveloped portion of the waterfront; or (b) is incorporated in the development of a sensitive structure such as a pile-supported wharf. In the case of piles, pipelines, or utility lines embedded in the embankment, the allowable deformation of these components will dictate the ground movements that can be tolerated. Post-earthquake serviceability requirements of the waterfront components that are founded on or near the embankments should guide the specification of seismic performance requirements of the embankments.

In addition to assessing the impact of embankment deformations on components in immediate contact with the retaining structure, the influence of the associated ground movements in the backland soils should also be considered. In several ports, efforts have been made to ensure that embankment deformations will remain small, although such efforts have not been made during the design of the cargo storage areas behind the embankments. Here, the soils are allowed to remain unimproved and potentially vulnerable to liquefaction. This is because any liquefaction and associated differential settlement and pavement damage that occurs in these backland areas would not suspend port operations, and regrading could be carried out quickly and relatively inexpensively.

The design of gravity breakwaters includes the bearing capacity of the foundation soils, settlement of the foundation soils due to consolidation, and stability due to wave loading. Experience at numerous ports around the world has demonstrated that the primary modes of failure due to seismic loading are foundation failures and excessive settlement. When this occurs, composite breakwaters may retain their vertical orientation, yet become submerged due to densification and deformations of the foundation soils. Given that the freeboard of a breakwater is the key issue, the performance requirements should focus on the potential for deformations of the foundation soils.

Guidelines for Preliminary Seismic Vulnerability Assessment

Preliminary seismic vulnerability assessment of existing embankments should be based on visual observation in the field and review of relevant office documents. Visual observation of embankments, dikes, and bulkheads is difficult because of their limited accessibility (i.e., they are buried or are commonly covered by soil layers or pavements). Breakwaters pose additional difficulties, since their offshore location limits direct

viewing at the dredge line. Nevertheless, visual observation is an important basis for assessing the integrity of these embankments. Visual evidence of foundation degradation, excessive settlements, etc. often indicates conditions that decrease the seismic stability of embankments. In particular, the engineer performing the visual observation should be aware of the following factors that could indicate a potential for poor seismic performance of embankments:

- Any observed undermining of foundation soils around breakwaters or bulkheads due to scour represents a possible location of large deformations during an earthquake. Inspection methods for scour include surveys by divers, or profiling techniques such side-scan sonar.

- Slumping of earth embankments due to washing of soils from behind armor layers can compromise the surficial layers of dikes and affect adjacent structures. Ground cracks caused by embankment settlements are evidence of weak, compressible foundation soils. Recurring tension cracks in backland areas indicate marginal static stability and significant vulnerability to earthquake-induced deformations. This global type of movement can also be indicated by deformations of the piles beneath wharf decks that are embedded in the embankment, and the misalignment of crane rails.

The office documents reviewed during a preliminary seismic vulnerability assessment of an embankment should include geotechnical reports, construction documents, as-built records, maintenance reports, etc. This review should focus on the seismic design provisions adopted, if any, and the construction methods used to place backland fill soil. Evaluations of the potential for liquefaction of the foundation soils and backfill, as well as global stability of the embankment should be emphasized. Maintenance reports for waterfront areas can provide evidence of in-stability of embankments, and may help to prioritize areas of the port for seismic retrofit.

A notable example of the possible benefits of preliminary pre-earthquake seismic vulnerability evaluation of embankments is the Port of Valdez, Alaska. In the two-to-three decades preceding the 1964 Alaska earthquake, pile-supported wharves embedded in gravel fill dikes suffered several failures under static loading conditions. Some of the failures were induced by cargo loads transmitted to weak foundation soils, while others were due to unstable slopes which may have failed in response to uncommonly great tidal fluctuations. These factors may have contributed to the catastrophic flow slide that occurred at the Port of Valdez during the 1964 Alaska Earthquake , which included almost 1200 m of shoreline at the port and claimed 30 lives. While this is an extreme example of what can happen, the potential for poor seismic performance that may be indicated by such pre-earthquake observations should be kept in mind during preliminary seismic vulnerability evaluations of port waterfront embankments.

Guidelines for Seismic Analysis

A seismic analysis of a port waterfront embankment should focus on two issues; (a) the stability of the embankment itself, and (b) the global stability of the embankment, backfill, and foundation soils. In most cases, common pseudostatic rigid body methods of evaluation will suffice for evaluating the stability of the embankment. These methods of evaluation are well established in the technical literature (e.g., Ebeling and Morrison, 1993; Kramer, 1996). However, although pseudostatic methods are useful for approximate analysis of seismic stability, they suffer from the following limitations: (a) they do not indicate the range of embankment deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils can only be approximated; and (c) coupled analyses that account for such factors as the degradation of soil strength and soil-structure interaction are not possible. Therefore, for those embankments where damage could lead to unacceptable risks to port operations, more refined analysis procedures that are summarized below should be used.

Enhancements to traditional pseudostatic limit equilibrium methods for estimating embankment deformations and the degradation of soil strength due to liquefaction or collapsible soil behavior have been proposed by numerous investigators (e.g., Makdisi and Seed, 1978; Byrne et. al., 1994). These methods are based largely on rigid sliding-block methods, wherein a portion of the embankment slides in response to ground motions that exceed a critical acceleration.

In situations involving pile supported structures embedded in dikes, or other embankment and structure deployment where soil-structure interaction effects could be significant, it is becoming more common to rely on numerical modeling methods to ascertain the likely range of embankment deformations during design-level earthquakes. Two-dimensional numerical models such as FLUSH (Lysmer et. al., 1975), FLAC (Itasca, 1995), and DYSAC have been used to model the seismic performance of waterfront components at ports (e.g., Werner and Hung, 1982; Roth and Inel, 1992; Dickenson and McCullough, 1998). These numerical analyses models differ primarily in the soil models employed and in their ability to model permanent deformations. Each has been useful in evaluating various aspects of dynamic soil-structure interaction.

Guidelines for Seismic Design of New Embankments

The seismic design of new embankments structures should address: (a) static stability issues (e.g., bearing capacity, sliding, forces due to wave loading, and (b) dynamic loading considerations that include the influence of inertial body forces on overall stability, the dynamic behavior of embankment and foundation soils, and soil-structure interaction effects. Also, the heights of breakwaters must be specified to account for consolidation settlements that may occur. As previously discussed, these methods of analysis include standard pseudostatic rigid-body analyses, non-coupled analyses (which

account for the loss of soil strength and stiffness as well as permanent deformations), and advanced numerical modeling techniques. The allowable deformations of the embankments and adjacent soils will reflect the importance of the components along the waterfront, which may vary at specific sites within individual ports.

During the seismic design process, the presence of any potentially liquefiable materials in backfill areas must be fully analyzed and expected settlements computed. Specific attention should be paid to the acceptability of the amount of settlements which can be tolerated, which will depend on the type and importance of the port operations in the vicinity Under a Level 1 seismic design, large deformations resulting in widespread pavement disruption should be avoided where economically feasible. In a Level 2 design, larger deformations of the embankment may be permitted, as long as the duration and costs of disruptions to the surrounding area are within acceptable limits and consistent with performance goals.

For Level 1 seismic design, the Factor of Safety against liquefaction in the backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.) For the Level 2 design, the Factor of Safety against liquefaction in the backfill should ***ideally*** be 1.0 or higher with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated.

If potentially unstable foundation soils are identified during the design phase of development, remedial strategies such as soil replacement (key trenches with engineered fill) or soil improvement may be used. Ground treatment can be carried out concurrently with reclamation and construction of the embankments, resulting in an expedient construction sequence.

Guidelines for Seismic Retrofit of Existing Components

Experience has demonstrated that, even at modern ports, embankments have been susceptible to earthquake-induced damage. Soil liquefaction and insufficient stability of the underlying foundation soils repeatedly appear as the predominant causes of earthquake-induced failures of existing embankments and associated damage to waterfront components. Seismic retrofit of embankments may include remedial measures to the embankment, to the foundation soils, or both depending on the results of seismic stability analyses. In the case of a marginally stable embankment founded on competent soils, remedial measures may include one or more of the following:

- Modifying the geometry of the embankment, either with berms or by reducing slope angles.

- Improving the strength of the soils by using mechanical densification, soil replacement, or cementation techniques.

- Strengthening the embankment through the use of structural stabilization techniques such as mixed in-place soil-cement walls, drilled piers, and driven displacement piles adjacent to the toe of the embankment. The latter method should be used with caution, since construction-induced vibrations can lead to excessive deformations of marginally stable embankments.

In many instances the foundation soils beneath the embankment are unstable under seismic loading. Soil improvement techniques can be used to mitigate these hazards. From a practical perspective, guidelines for specifying the volume of soil to be treated and the degree of improvement required to insure that earthquake-induced embankment deformations are held to within allowable limits have not been well developed. Recommendations have been provided (e.g., PHRI, 1997) but very few case histories exist for evaluating the performance of embankments that have undergone soil treatment. The limits of soil treatment are usually determined by performing a series of sensitivity analyses wherein the width of the improved soil zone is related to either the computed factor of safety against sliding or the estimated deformations. The requisite volume of soil improvement will reflect site specific factors such as the embankment type and geometry, depth of weak soils, density of the backland soils adjacent to the structure, as well as the characteristics of the design level ground motions.

The constructability of these remedial strategies is complicated by the location of preexisting port components such as piles, pipelines, above- and below-ground utility conduits and overhead structures. In addition to limiting access and constraining the locations of work platforms, existing buried components can be adversely affected by several of the ground treatment methods. Soil densification techniques that rely on vibration (e.g., vibro-compaction, deep dynamic compaction or soil displacement (compaction grouting) increase lateral earth pressures. The potential for this increased pressure should be acknowledged when improving soils in close proximity to buried structures.

It should be noted that there may be a need for a seismic reassessment following an earthquake. This reassessment should be triggered when excessive deformations are observed. The decision process depends on the specifics of the geometry and soils present. Typical soil limit deformations are given in following sections.

Gravity Retaining Structures

Types of Gravity Retaining Structures

Gravity earth-retaining structures are widely used along the waterfront for quay walls, sea walls, and lock and dam structures Numerous wall types and wall geometries

have been employed at ports (Figure 2-3). A broad categorization of the most common retaining structures is provided below.

Concrete Block Walls These structures are composed of smooth or interlocking blocks that are stacked one on top of another to the design height. Pile foundations are used in regions with weak foundation soils or other areas with a limited supply of suitable fill for key trenches. These walls can be either vertically faced or stepped to slope at specified angles. The primary advantages of these walls include: (a) durability to environmental agents and impact by vessels; (b) good quality control achieved during fabrication; (c) simple construction; and (d) adaptability to a variety of foundation conditions.

The basic design of block-work walls can be generally classified as follows: (a) bonded construction using solid concrete blocks; and (b) walls formed with hollow or special concrete blocks (Tsinker, 1997). The type of bonding between the blocks will influence the seismic performance of the walls, since sliding may tend to occur at the interface between adjacent blocks during shaking.

Concrete Caissons The two most predominant types of concrete caissons used at ports are "box-type" cassions and counterfort caisson walls. These structures are built onshore, transported to the waterfront and sunk into position. Box-type caissons can be floated into place. Pile foundations are often used in weak soils when suitable replacement soils are not readily available. In the case of box-type caissons, internal walls in the caisson form cells that are filled with granular material (e.g., soil, slag, concrete construction debris) or water, depending on the lateral earth pressures that must be resisted and the allowable bearing pressures on foundation soils. The caissons are usually placed on a prepared foundation pad of granular fill and backfilled with sand or rubble. The density of the foundation and backfill soils will have a significant effect on the seismic performance of the caisson.

Cellular Sheet Pile Structures Cellular steel sheet pile bulkheads are usually constructed from flat web sheet piles that are driven with vibratory equipment. The shape of the cell is maintained during the construction process with the use a of template for guiding the sheet piles during placement. Arc sections are driven on one or both sides of the bulkhead, and granular soils are used to fill the cells. The fill soil is often densified to increase the lateral stability of the cellular bulkhead. This is advantageous for the seismic performance of the bulkhead and the densification also reduces the liquefaction susceptibility of the fill.

These earth retaining structures differ from the others listed in this section in that they are flexible. This flexibility provides some reduction in the dynamic earth pressures experienced during earthquakes, although progressive permanent deformations can occur. Excessive deformations can lead to high interlock tension and potential failure. Liquefaction of the interior fill will also result in excessive interlock tension. The failure of a 45-year-old sheet pile cellular bulkhead occurred due to liquefaction generated by ground motions of moderate intensity (PGA approx. 0.15 g to 0.20 g). Post-earthquake investigations of this facility revealed that corrosion of the sheet piles and improper placement of the sheet piles had contributed to the failure. Standard methods of analysis, including pseudostatic seismic design, are presented by Schroeder (1990).

Steel Plate Cylindrical Caisson. Large diameter cylindrical caissons have been used in numerous ports. The primary advantages of this caisson type over comparable sheet pile structures are: (a) the cell is fabricated onshore; and (b) placement is much faster than with cellular sheetpile bulkheads. However, the cylindrical caisson does require the preparation of a reasonably flat bedding pad. The steel cell is placed by sinking it into place and embedding it into the soil through driving with vibratory hammers. In most cases, steel arc sections are then placed on both sides of the wall and joined to the cylindrical caisson by interlocks.

Cribwork Quay Walls Crib walls at ports have been constructed from timber cribwork and concrete cribwork. Cribs are rather labor intensive, and they must be constructed onshore and launched and sunk into position. They are then filled with gravel or rock fill to form a gravity structure. The crib wall can either be full height or provide a supporting base for mass concrete superstructure walls which are placed on the cribwork (Tsinker, 1997).

General Functional/Operational Requirements at Ports

The primary operational requirement of gravity retaining structures is to resist lateral earth pressures with minimal deformation. These structures resist the lateral earth pressures by virtue of their body weight and the resulting frictional resistance mobilized between the structure and the foundation soil. These massive structures require strong foundation soils and it has often been necessary to enhance the bearing capacity of the foundation by excavating trenches and replacing the weak soils with cohesionless fill, or by supporting the structures on piles or pile supported relieving platforms. Specifications for allowable wall-backfill deformations will be based on the sensitivity of the structures located in close proximity to the retaining structures.

Guidelines for Developing Seismic Performance Requirements

Where gravity retaining structures are deployed alongside key waterfront cargo handling operations, the seismic stability of such structures is a major concern. Permanent deformations of the retaining structures and surrounding soils must be minimized to

ensure serviceability following design-level earthquake motions. Therefore, it follows that the seismic performance requirements for gravity should focus establishment of allowable wall deformations under the design-level earthquake motions such that: (a) the operations of key components that are supported on the retaining structures will not be adversely affected; and (b) associated ground movements in the backland soils will lead to acceptable levels of damage to the structures and cargo storage facilities in those areas.

In general, waterfront retaining walls should perform to the following standards;

1. To resist earthquakes of moderate size, Level 1, which can be expected to occur one or more times during the life of the structure without significant damage (i.e. displacement). As a general guideline, the deformations associated with this performance requirement are roughly 1 inch or less of settlement and lateral deformations of about 3 inches or less.
2. To resist major earthquakes, Level 2, which are considered infrequent rare events maintaining life safety and precluding total collapse, but allowing a measure of controlled inelastic behavior which will require repair. The allowable deformations for Level 2 earthquakes are a maximum of 4 inches of settlement and lateral deformations of about 6 to 12 inches or less.

In order to ensure that the approximate deformation limits are not exceeded, liquefaction hazards must be fully evaluated. Specific attention is to be paid to the acceptability of the amount of settlements. Under Level 1 earthquake motions, large deformations resulting in widespread pavement disruption should be avoided where economically feasible. At several ports, liquefaction mitigation efforts have focused on limiting earthquake-induced wall deformations and applying only nominal soil improvement in backland areas such as cargo storage yards located well behind the walls. Although liquefaction in these areas would result in differential settlements and damage to pavements, these effects would not suspend port operations and regrading could be carried out quickly.

Guidelines for Preliminary Seismic Vulnerability Assessment

Preliminary assessment of the seismic vulnerability of existing gravity retaining structures should be based on field inspections and a review of design and construction documents. Field inspections of retaining walls are onerous due to the development of the waterfront around the structure. The waterfront location of the walls and their partial burial with backfill will conceal them from direct view. In addition, retaining walls are commonly covered with pavements and structures. However, inspections can provide evidence for of existing conditions that could lead to poor seismic performance during future earthquakes such as degradation of the structure, evidence of ground movement under static conditions, and other adverse conditions such as excessive scour beneath the gravity wall. This is especially true for steel sheet pile bulkheads which are prone to

corrosion. Evidence of foundation degradation, excessive settlements, etc. is often indicative of conditions that decrease the seismic performance of retaining structures. The following is a partial list of potential factors that could adversely affect the seismic performance of waterfront gravity walls, the following have been observed at ports:

- Undermining of foundation soils around quay walls due to scour. Inspection methods for scour include surveys by divers or profiling techniques such side-scan sonar.

Slow, yet continuous deep-seated rotation due to the marginal bearing capacity of foundation soils. This global type movement can also be indicated by rotation of the gravity walls, deformations of relieving platforms, persistent tension cracks in backland pavements, and the misalignment of crane rails or utility lines that are supported on the gravity wall. These observations provide evidence low static stability and significant vulnerability to earthquake-induced deformations.

An important aspect of the vulnerability assessment should also include a thorough review of office documents (e.g., geotechnical reports, construction documents, as-built records, and maintenance reports). This review should focus on the seismic design provisions adopted, if any, and the construction methods used to place backland fill soil. Evaluations of the liquefaction susceptibility of foundation soils and backfill, as well as global stability of the retaining wall should be emphasized. Maintenance reports for waterfront areas can provide evidence for marginal stability of retaining walls and may help to prioritize areas of the port for retrofit strategies.

Guidelines for Seismic Analysis

Seismic analyses for new and existing retaining structures must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the walls. Potential failure modes include: sliding, overturning (for rigid walls only), bearing capacity failure, and deep seated instability.

The most commonly used seismic design methods for gravity structures are based on standard pseudostatic limit equilibrium methods of analysis wherein a static horizontal seismic coefficient is applied as an additional body force (Ebeling and Morrison, 1993). The pseudostatic method of analysis suffers from two significant deficiencies when applied to waterfront retaining structures: (a) the loss of soil strength associated with the generation of excess pore pressures during earthquakes can only be approximately accounted for using post-liquefaction residual undrained strengths for the sandy soils; and (b) the deformations of the wall and adjacent soil can not be evaluated. The limitations imposed by these design methods can be significant in light of the role that liquefaction plays in the seismic performance of waterfront retaining structures.

As a means of estimating earthquake-induced deformations of gravity walls, limit equilibrium analyses can be supplemented with rigid body, sliding block-type displacement analyses which are used to estimate the seismically induced movement of

retaining walls acceleration. This method of analysis is similar to the procedures for analysis of earthquake-induced deformations of slopes previously summarized. In this method, the lateral acceleration that yields a factor of safety against sliding equal to unity is defined as the critical (or yield) acceleration. A suite of appropriate acceleration time histories is then used in conjunction with the critical acceleration and the permanent displacements calculated. This technique has been used as the basis for several common methods that have been developed for the estimation of gravity wall displacements (e.g., Richards and Elms, 1979; Elms and Richards, 1990; Whitman and Liao, 1985).

The allowable deformations of the retaining structure should reflect the impact that the deformations have on the stability of the wall, and as well as the sensitivity of nearby waterfront components to lateral and vertical deformations of the retaining structure. Key considerations pertinent to the specification of allowable deformations may include: (a) whether the crane rails are tied together; (b) whether utility conduits are rigidly fastened to or pass between construction joint in the retaining structures; (c) whether pile supported structures are connected to the retaining structure and, if so, the ductility of these connections.

In projects involving displacement-sensitive retaining walls, advanced numerical modeling techniques are recommended for estimating permanent displacements due to earthquakes. The primary advantages of these models include: (a) complex wall geometries can be evaluated; (b) sensitivity studies can be readily performed to estimate the influence of various parameters on the seismic stability of the retaining structure; (c) dynamic soil behavior is much more realistically reproduced; (d) coupled analyses can be used that account for such factors as excess pore pressure generation in contractive soils during ground shaking and the associated reduction of soil stiffness and strength; (e) soil-structure interaction effects and permanent deformations can be evaluated.

Despite the above advantages of numerical modeling, several practical issues may limit the utilization of this analysis tool. These concerns include; (a) the engineering time required to construct the numerical model can be extensive for complex geometries; (b) numerous soil parameters are often required, thereby increasing the cost of geotechnical investigations; and (c) because very few of the available models have been validated with well-documented case studies of the seismic performance of actual retaining structures, the level of uncertainty in the analysis is often difficult to assess.

Experience demonstrates that the primary source of damage to waterfront retaining structures is liquefaction of sandy soils in the backfill, foundation, and below the dredgeline in front of the walls. Therefore, the presence of any potentially liquefiable soils should be fully analyzed and expected settlements should be computed. In many cases, remedial ground treatment will be required to increase the liquefaction resistance of the soils. Along these lines, it is noted that an important benefit of the advanced numerical modeling tools discussed above is their ability to assess the relative effectiveness of alternative methods and extents of soil improvement in reducing the potential for liquefaction and improving the seismic performance of the retaining structure.

Guidelines for Seismic Design of New Components

Current standards of practice for the seismic design of gravity retaining structures are well documented in a number of very useful and up-to-date manuals and textbooks - (e.g., Ebeling and Morrison, 1993; Kramer, 1996). These methods commonly use rigid-body limit-equilibrium methods of analysis which offer the following practical advantages: (a) the techniques are familiar to most engineers; (b) requisite input includes standard geotechnical parameters that are obtained during routine foundation investigations; and (c) the methods have been coded in very straight-forward and efficient computer programs that facilitate the performance of sensitivity studies for various design options.

Widely-used limit equilibrium methods of analysis require that potential seismically-induced movement of the wall be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). Determining the lateral earth pressures acting on the retaining structure is a necessary first step in the stability analyses. To estimate the dynamic lateral earth pressures, a static body force representing the inertial effects imposed by the ground motions must be added to the wall and backfill soil. Seismic design factors in the form of the pseudostatic seismic coefficients (k_h and k_v) are determined as a fraction of the maximum peak accelerations generated by the design earthquake motions. For retaining structure design, the seismic coefficients are commonly specified as one-third to one-half of the peak horizontal ground surface acceleration (pga). The standard that has been adopted in Japanese practice relates the horizontal seismic coefficient to the peak surface acceleration in the following relationship:

$$k_h = (pga/g) \qquad \text{for pga} < 0.2\text{ g}$$

$$k_h = 1/3\ (pga/g)^{0.3} \qquad \text{for pga} > 0.2\text{ g}$$

Limit equilibrium analyses can be performed as standard pseudostatic analysis (e.g., Mononobe-Okabe method as described by Whitman and Christian, 1990), or in rigid body, "sliding-block" type displacement analyses (e.g., Elms and Richards, 1990 or Whitman and Liao, 1985) which are used to estimate the seismically induced movement of the retaining structure. The specification of allowable wall deformations should take into consideration the displacement sensitivity of appurtenant structures and adjacent components.

Guidelines for Seismic Retrofit of Existing Components

In seismically active regions of the world, one the most pressing issues at ports is the anticipated seismic performance of existing gravity retaining structures. In many cases, existing structures have performed poorly for one or more of the following reasons:

(a) inadequate height-to-width ratios due to the use of low seismic coefficients in original design; (b) the presence of weak foundation soils which could lead to deep-seated foundation failures; and (c) use of fill placement methods during initial construction that have resulted in loose soils that are prone to liquefaction.

Assessment of this seismic performance of existing gravity retaining structures subjected to their design-level ground motions should be evaluated using appropriate seismic analysis procedures, together with liquefaction hazard analysis procedures. Depending on the results of these analyses, appropriate seismic retrofit methods may be implemented. The most common retrofit methods include: (a) implementation of anchor systems for the gravity structure; (b) augmentation of the wall to increase its cross sectional area; (b) construction of a new wall outboard of the existing structure; or (c) replacement of the wall. Any of these methods should be supplemented with soil improvement of the surrounding fills, because of the demonstrated effectiveness of soil improvement methods in improving the seismic performance of gravity retaining walls during past earthquakes.

Soil improvement techniques have been used to mitigate liquefaction hazards to waterfront retaining walls at numerous ports throughout the world (e.g., Iai et. al., 1994). All other factors being equal, the effectiveness of the soil improvement is a function of the level of densification and the volume of soil that is treated. Although few case histories exist for the performance of improved soils subjected to design-level earthquake motions, experience has shown that caissons in improved soils have performed much more favorably than have adjacent caissons at unimproved sites which experienced widespread damage (e.g., Iai et. al., 1994).

The Japan Port and Harbour Research Institute (PHRI, 1997) has produced one of the few design guidelines that exist for specifying the extent of soil improvement adjacent to waterfront retaining structures. The recommended extent of ground treatment is shown in Figure 2-4. The stability of the caisson is evaluated using standard limit equilibrium methods in which a dynamic pressure and a static pressure corresponding to an earth pressure coefficient K=1.0 are applied along plane CD due to liquefaction of the unimproved soil. These guidelines for establishing the soil improvement area and evaluating caisson stability are valuable design tools, however, they do not address the seismically-induced deformation of the caisson and backfill soils.

In order to develop a simplified technique for estimating seismically-induced deformations of gravity caissons Dickenson and Yang (1998) have developed simplified design charts for the application of soil improvement adjacent to gravity retaining walls. The authors utilized a numerical model, validated with well-documented case histories, for parametric studies of caisson performance. The results of the study have been synthesized into practice oriented design charts for estimating the lateral deformations of gravity quay walls.

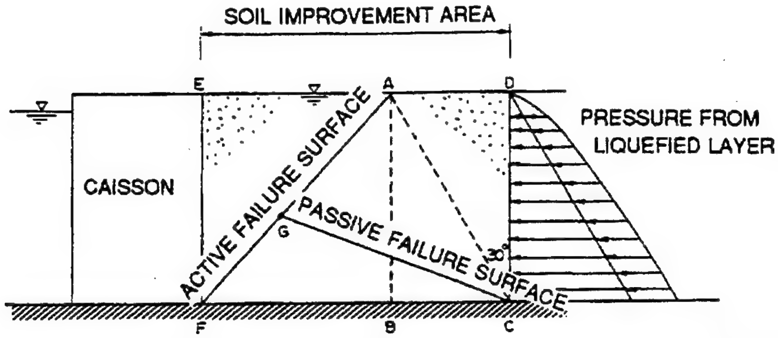

Figure 2-4: Schematic diagram for investigation of stability with respect to pressures applied from the liquefied sand layer (PHRI, 1997)

The results of the parametric study demonstrate the influence of ground motion characteristics, geotechnical parameters, and caisson geometry on the deformations of the retaining walls. These results have been synthesized into normalized parameters, where possible, to incorporate the key variables into straightforward design parameters. For example, the wall geometry has been expressed by W/H ratios as previously mentioned, the width of the zone of soil improvement is given as a function of the height of the wall (L/H). In order to account for the duration of the earthquake motions a normalized ground motion intensity has been used. This parameter is defined as the maximum horizontal acceleration at the top of the dense soil $(A_{max})_D$ divided by the appropriate magnitude scaling factor (Arango, 1996). The magnitude scaling factors are provided in Table 2-1. It is recommended that if a site specific seismic study is not performed to determine $(A_{max})_D$, then the peak ground surface acceleration can be reduced using the reduction factor (r_d) developed for estimating the variation of cyclic shear stress (or acceleration) with depth (Seed and De Alba, 1983). The values of r_d for 15 and 7.5 meter walls are approximately 0.78 and 0.95, respectively. It should be noted that the reduction factor was developed using one-dimensional dynamic soil response methods and this will yield approximate acceleration values for the two-dimensional soil-structure interaction applications discussed herein.

Table 2 –1. Magnitude Scaling Factors Derived by Arango (1996)

Earthquake Magnitude	8.25	8	7.5	7	6	5.5
MSF	0.63	0.75	1	1.25	2	3

The results of the parametric study are shown in Figure 2-5. The normalized lateral deformations at the top of the wall, X_d/H, are plotted versus the normalized width of the improved soil, L/H, and as functions of backfill density and the W/H ratios of the caissons. The numbered triangles superimposed on the charts correspond to field case histories. In this figure, the rubble fill adjacent to the caissons has been treated as non-liquefiable soil, thereby contributing to the "effect" width of the improved (i.e., non-liquefiable) soil. In the case of triangular, single-lift sections of rock fill the width of the

rubble fill has been approximated as one half of the width of this fill at its base. The relationships provided in Figure 2-5 clearly demonstrate the benefit of ground treatment on the seismic performance of the caissons. It is also evident that the incremental benefit of a wider zone of ground treatment begins to decline once the soil improvement extends more than about 2.0 to 3.5 times the total height of the wall. At this point the cost of additional soil improvement may outweigh the benefits. It is interesting to note that the soil improvement guidelines prepared by the PHRI (1997) correspond to a normalized width of soil improvement of roughly 1.3 to 1.6, as supported by the work of Iai (1992).

As a screening tool for estimating the seismically-induced displacements of caissons, the recommended procedures for utilizing the results of the parametric study include:

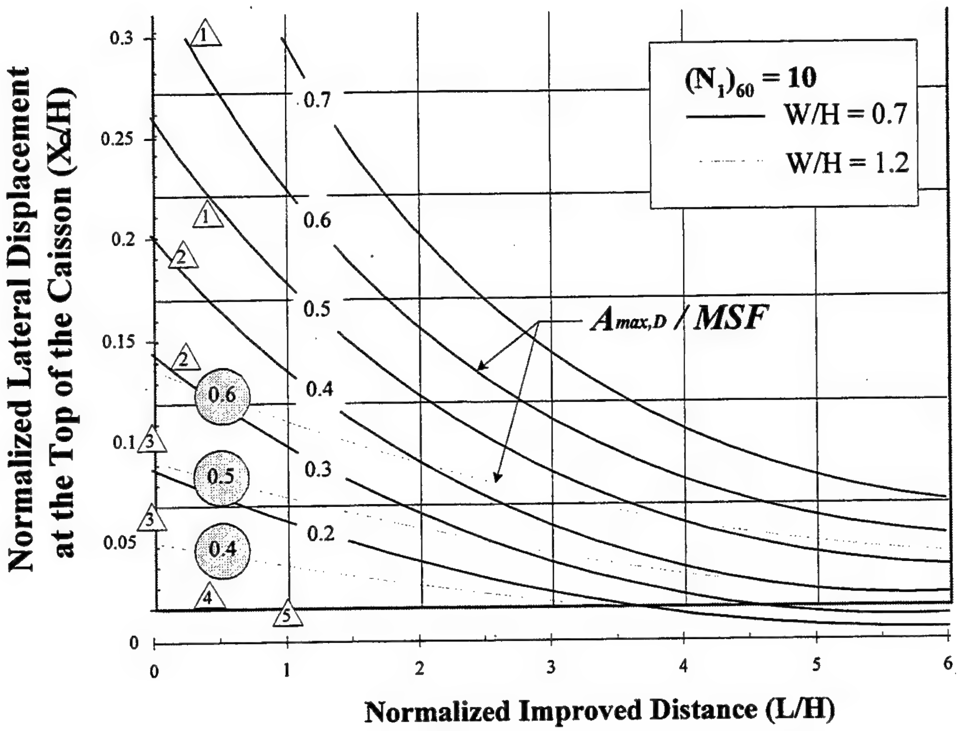

Figure 2-5(a): Normalized lateral displacements for backfill $(N_1)_{60}$=10

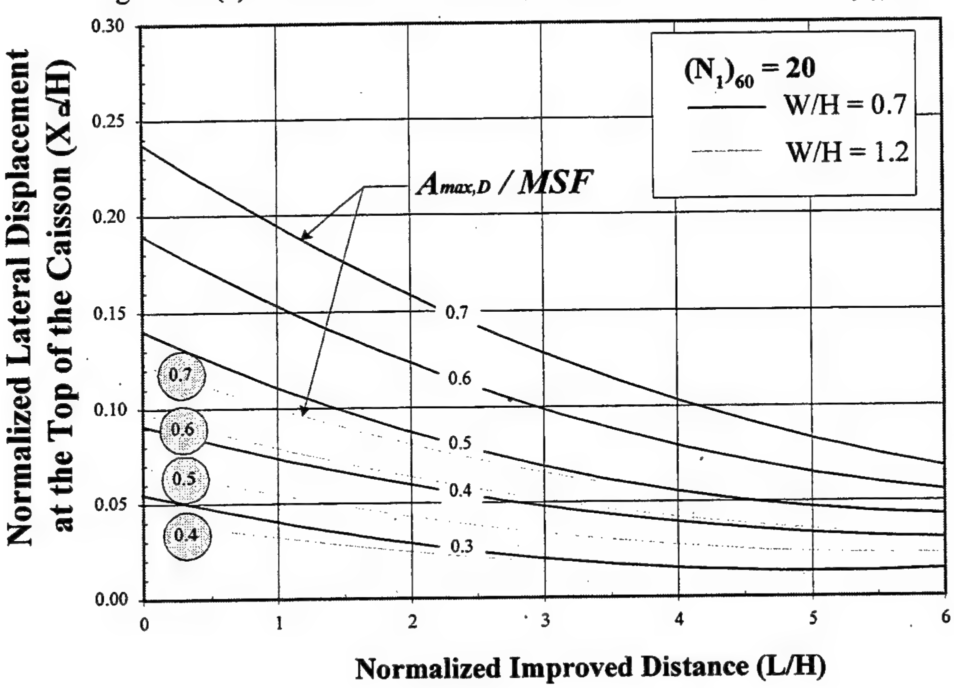

Figure 2-5(b): Normalized lateral displacements for backfill $(N_1)_{60}$=20

Figure 2-5 Design Charts for Estimating Lateral Displacements of Caissons.

1. Design the wall using standard pseudostatic limit equilibrium methods to determine the wall geometry (W/H).

2. Determine $(A_{max})_D$ based on a site response analysis or approximate with empirical soil amplification factors to yield the peak ground surface acceleration and the reduction factor (r_d).

3. Select the magnitude scaling factor (MSF) for the specified earthquake magnitude, and compute the ground motion intensity factor as $(A_{max})_D$/MSF.

4. Given the standard penetration resistance of the backfill soils, the width of the ground treatment behind the caisson, and the ground motion intensity factor, enter Figure 2-5a or 2-5b and obtain the normalized lateral displacement. From this, the deformation at the top of the wall (X_d) can be estimated.

When it is shown to be impossible to or uneconomical to achieve the levels of performance associated with new components, an acceptable risk assessment (including economic life cycle cost analysis) should be performed to establish the most appropriate performance level form a cost-benefit standpoint.

The critical role of soil liquefaction in most earthquake-induced waterfront retaining wall failures requires that this seismic hazard be evaluated for the backfill and foundation soils. Under Level 1 earthquakes large deformations resulting in widespread pavement disruption should be avoided where economically feasible. The following guidelines for both new and existing facilities have been recommended for use at U.S. Navy facilities (Ferritto, 1997a):

- For a Level 1 earthquake the Factor of Safety against liquefaction in the backfill should be 1.5 or higher with settlements of about 1 inch or less and lateral deformations of about 3 inches or less.
- For a Level 2 earthquake the Factor of Safety against liquefaction in the backfill should be 1.0 or higher with settlements of about 4 inches or less and lateral deformations of about 6 to 12 inches or less. Where it may not be possible to achieve a Factor of Safety greater than 1.0, a Factor of Safety greater than 0.9 may be considered as long as the computed deformation state is shown to have limited controlled settlements and lateral spread equivalent to the values stated.

Steel Sheetpile Wharves

Types of Anchored Steel Sheet Pile Bulkheads

Steel sheet piles bulkheads function as earth retention systems, berthing structures, flood walls, and sea walls. These structures have been used where soil conditions permit driving of these relatively flexible piles. Common configurations

include: (a) relatively short cantilever sheet pile walls, which derive support solely through pile stiffness and passive soil resistance beneath the dredge line; and (b) taller anchored sheet pile structures, which are supported by tie rods (at one or more elevations) fixed to mechanical anchors. The combination of weak soils commonly found in the marine environment and the wall heights required at berths precludes the use of cantilever walls in most waterfront applications. This subsection addresses seismic performance issues specifically associated with anchored sheet pile bulkheads.

A variety of anchored wall configurations have been used in the development of marginal wharves at ports (Figure 2-6). The more common configurations are briefly summarized below.

Sheet Pile Wall with Deadman Wall Anchorage. As shown in Figure 2-6, lateral support for the sheet pile wall can be provided by tie rods that extend to concrete blocks (deadman) or a continuous wall. Tie rod spacing will reflect factors such as wall height, soils in the backfill and foundation below the dredge line, and wall stiffness.

Sheet Pile Wall with Batter Pile Anchorage. In situations where adequate lateral restraint cannot be provided by a deadman, an anchor system made up of piles can be used. This is common for tall walls in relatively weak soils where the lateral earth pressures that must be resisted by the bulkhead would exceed the passive resistance provided by shallow anchor blocks. Single vertical piles, small pile groups, and batter piles have been used as effective anchors. Batter piles offer the advantage of increased lateral resistance due to their orientation relative to the wall, although their very stiff connection at the pile cap lead to problems during an earthquake.

Double (Paired) Sheet Pile Walls In regions where steel sheet piles are readily available, it is often beneficial to forego batter pile anchorages for a sheet pile wall support system. The sheet pile anchor wall is usually constructed with the same materials and geometry as the bulkhead, but is usually much shorter. The paired walls are connected with same wale and tie rod arrangement used for other anchored walls.

General Functional/Operational Requirements at Ports

The primary operational requirement of sheet pile bulkheads is that they resist lateral earth pressures with minimal deformation. These requirements are the same as those for gravity retaining structures.

Guidelines for Seismic Performance Requirements

The seismic performance requirements for anchored sheet pile bulkheads are essentially the same as the requirements for gravity retaining structures previously discussed. Briefly summarized, the design of anchored sheet pile bulkheads should limit permanent lateral displacement at the top of the sheet pile to values that are based on the

displacement tolerances of the key port components in the vicinity of the sheet pile structure. For example, the limiting displacement criteria for sheet pile bulkheads at U.S. Navy ports is as follows (Ferritto, 1997a): (a) under the Level 1 ground motions, the permanent lateral displacement at the top of the bulkhead must be less than 1 in.; and (b) under the Level 2 ground motions this permanent lateral displacement must be less than 4 in. These values are presented as examples only, and different displacement values may be selected at a port, depending on the port's overall seismic performance requirements and those of the port components located near the bulkhead. The results of advanced numerical modeling of anchored sheet pile bulkheads commonly indicate that the 4-inch displacement limit used by the Navy for Level 2 earthquake motions cannot be met by standard bulkheads. In addition, in cases where the anchor does not experience catastrophic failure, the maximum displacements during the stronger earthquake motions do not occur at the top of the bulkhead wall, but instead occur between the elevation of the anchor and the dredge line. These lower displacements will still yield excessive ground surface deformations. These factors should be considered when establishing seismic performance requirements for sheet pile bulkheads.

Guidelines for Preliminary Seismic Vulnerability Assessment

Review of Design and Construction Documents As for other port waterfront structures, preliminary assessment of the seismic vulnerability of existing sheet pile bulkheads should be based on visual inspections and review of design and construction documents. Because of the lack of accessibility to underwater or underground elements of the bulkhead, many aspects of its seismic vulnerability can only be assessed through review of its design and construction documents. Documents that should be reviewed for sheet pile bulkheads are design drawings and calculations, soils reports, and construction documents.

The review of design and construction documents should focus on: (a) review of pertinent geotechnical reports and construction documents to identify the existence of if the soils below the dredge line, in the backfill, or in the foundation are potentially unstable; and (b) the review of the seismic design procedures (if any) that were employed during the design of the sheet pile bulkhead, to check whether the original design assumptions are consistent with current knowledge and practice. In this, special attention should be paid to the type of tie-rod\anchor system (e.g., tie rods and deadman, tie-rods and anchor wall, tiebacks with grouted anchor, tie-rods and pile anchors) and the location of the anchor. Field experience, supplemented by numerical analyses, demonstrate that the anchor must be located further behind the wall than that specified for static design. Also, tie-rod failures constitute one of the most common failure modes of sheetpile bulkheads during earthquakes; therefore reevaluations should focus on the dynamic forces expected during the design level earthquakes. As-built construction data should be reviewed to determine the capacity of the tie rods and the connections between the tie rods and the bulkhead.

Field Inspection Despite the above-indicated accessibility problems, visual inspection can still provide important information for assessing the seismic vulnerability of existing sheet pile bulkheads. For example, such assessments may uncover evidence of ground movement under static conditions in the vicinity of the bulkhead that could indicate a potential for poor performance during future earthquakes. They can also serve as a means for documenting the existence of any visible corrosion of the sheet piles, as well as the types of port components and utilities in the vicinity of the sheet pile structures. This can provide a basis for assessing whether the allowable ground displacement criteria previously established for the sheet pile bulkhead are consistent with the limiting displacement and deformation tolerances of these other components.

In cases where serious corrosion has been observed, it may be necessary to remove specimens of the sheet piles for inspection and testing to insure the integrity of the corroded sections. In some cases, evidence of severe corrosion of sheet piles beneath the waterline has been manifested as sinkholes at the ground surface adjacent to the structure, due to the loss of backfill soil through holes in the sheet piles. Divers and/or side-scan sonar techniques should also be used to facilitate underwater inspection of the sheet piles and the depth of potential scour along their face. Additional evaluation may involve excavating along selected portions of the bulkhead to reveal the integrity of the tie rod-wale connections.

Guidelines for Seismic Analysis

Seismic analyses for new and existing bulkheads must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the walls. Again, potential failure modes include: excessive deformation of the bulkhead, passive failure of soil in front of anchors, tie rod failure, wale system failure, loss of passive soil resistance beneath the dredge line, interlock failure between sheet piles, and global stability when founded on weak soils, and potential damage to very stiff batter pile supported anchors at the connection of the piles to the anchor.

Widely used limit equilibrium methods require that potential seismically-induced movement of the wall be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). It should be noted that the nonuniform deformations typical of flexible sheet pile structures are not strictly accounted for in standard rigid body, limit equilibrium analyses. This variation in soil deformation over the height of the wall results in a statically indeterminant problem. Approximate methods are nonetheless used to evaluate the static and dynamic performance of these structures (e.g., free-earth support and fixed-earth support methods of analysis). Recommended references for seismic analysis and design of sheet pile bulkheads are provided by; Ebeling and Morrison (1993), USACOE (1994), and Kramer (1996).

As with the gravity retaining walls, determining the lateral earth pressures acting on the retaining structure is a necessary first step in the seismic analysis of sheet pile

bulkheads. In order to estimate the dynamic lateral earth pressures, a static body force representing the inertial effects imposed by the backfill soil during the ground shaking must be applied to the bulkhead wall. and backfill soil. (Since the mass of the bulkhead is very small, the resulting inertia forces due to structure weight will be small when compared to the effective soil mass.) Seismic design factors in the form of the pseudostatic seismic coefficients (k_h and k_v) are determined as a fraction of the maximum peak accelerations generated by the design earthquakes. As for gravity retaining wall design, the seismic coefficients established for design of sheet pile bulkheads are commonly specified as one-third to one-half of the peak horizontal ground surface acceleration (pga). The appropriate ratio to use will reflect the specified factor of safety for stability and the allowable deformations (i.e., the smaller the allowable deformations the larger the lateral seismic coefficient). The pseudostatic forces computed using these seismic coefficients are applied to the body of soil behind the sheet pile bulkhead structure. Japanese standards for establishing these seismic coefficients are the same as previously described for gravity retaining structures.

There is extensive experience on the performance of anchored sheetpile walls. Extensive liquefaction of loose saturated cohesionless soils in the backfill have caused major failures. Typical failures take the form of excessive permanent seaward tilting with associated movement of the anchor block. Associated with this is the settlement and cracking of the backfill soil. Gazetas and others (1990) review procedures used to analyze quaywalls. Pseudostatic procedures are used to determine lateral earth pressures after the well known Mononobe-Okabe approach. Statistics show that performance of quaywalls over the last 45 years has not improved despite increases in the seismic coefficients and refinements in the design methods. The dominant factor in failures of these walls is the loss of strength of the backfill and foundation soils. The pseudo static method of analysis suffers from three significant deficiencies: the failure to account for the loss of strength associated with the generation of excess pore pressure, the overestimation of the passive soil resistance of the anchor, and the inability to include the deformation and movement of the wall and soil. Many designs underestimated the level of seismic exposure and the design procedure ignores the vertical component of acceleration. which can increase the effective acceleration relating to active and passive earth pressures.

Gazetas and others (1990) developed an empirical design chart based on numerous case studies of sheetpile walls at sites where liquefaction was not observed at the ground surface. This chart based screening tool can be used to enhance conventional pseudostatic procedures. A horizontal acceleration factor is defines as:

$$k_h = \frac{2a_h}{3g} \qquad 2\text{-}1$$

A vertical acceleration factor may be assumed as two-thirds of the horizontal.

$$k_v = 2/3\ (k_h) \qquad 2\text{-}2$$

An effective acceleration coefficient is defined as:

$$k_e = k_h / (1 - k_v) \qquad 2\text{-}3$$

For cohesionless soils under water, the value of k_e may be increased by 1.5 to account for the potential of strength degradation from porewater pressure buildup. Figure 2-7 shows the nomenclature used. Figure 2-8 shows relationships for the active failure surface inclination, α_{ae}, and the active and passive seismic pressure coefficients as functions of the effective acceleration. The effective anchor distance, EAI is defined as

$$EAI = d / H \qquad 2\text{-}4$$

Having the effective acceleration coefficient one may determine the failure surface inclination and the seismic pressure coefficients. A trial value of EAI may be selected and the anchor length determined using Figure 2-9 and:

$$EPI \approx \frac{K_{PE}}{K_{AE}} \left(r^2\ (r+1)\right) \qquad 2\text{-}5$$

where

$$r = f / (f + H) \qquad 2\text{-}6$$

$$L \geq (H + f)\ \mathrm{Cot}\,(\alpha_{ae}) + (EAI_c)\ H \qquad 2\text{-}7$$

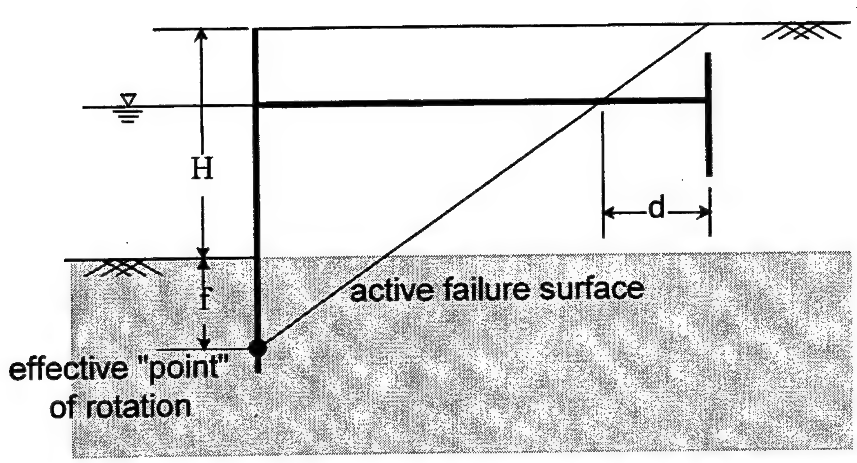

Figure 2-7: Definition of Effective Ancho Index : EAI = d/H (Gazetas et. al., 1990).

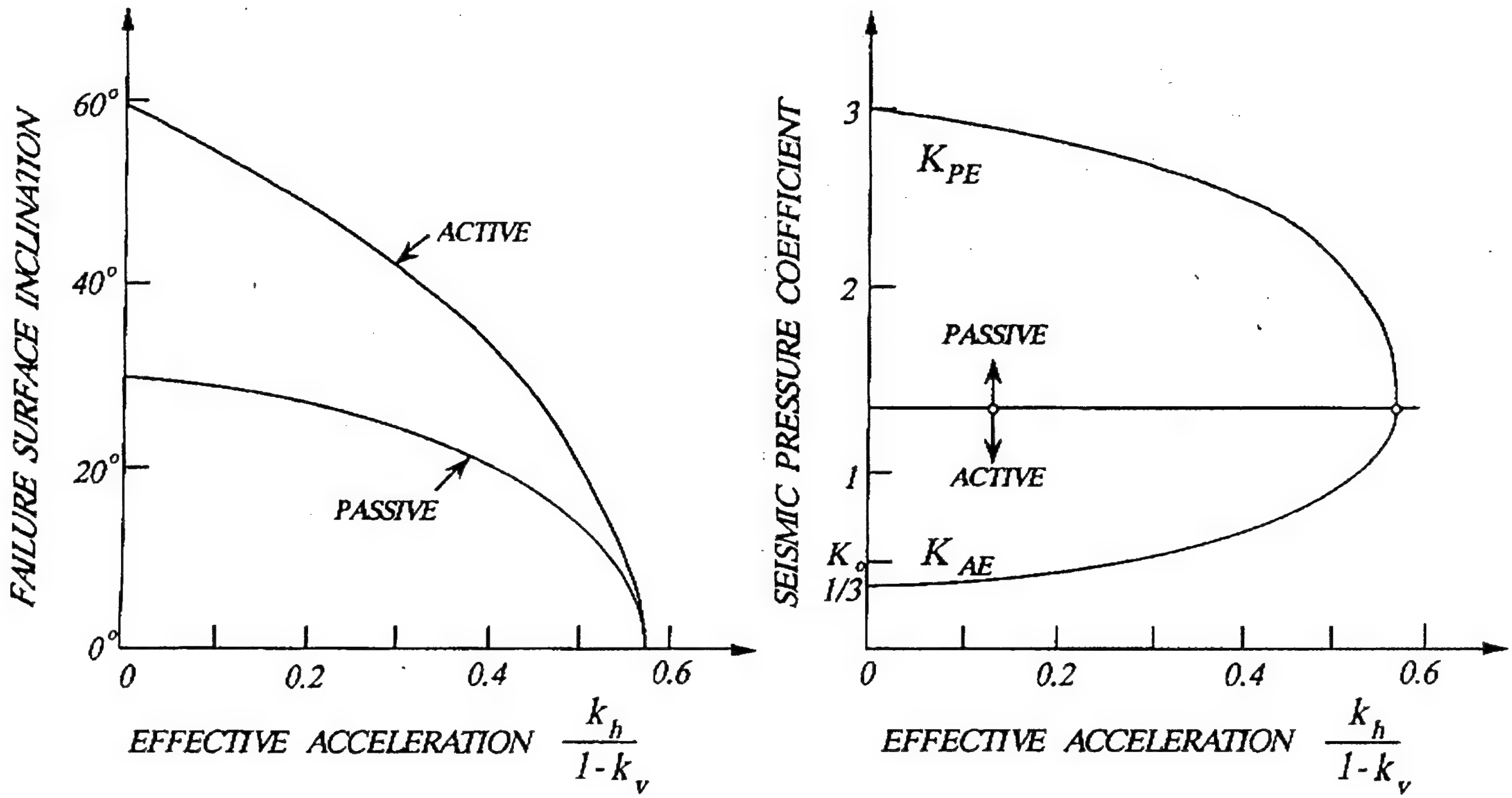

Figure 2-8: Effect of Horizontal and Vertical Seismic Coefficients on the Angle of the Active and Passive Sliding Wedges (Gazetas et. al., 1990).

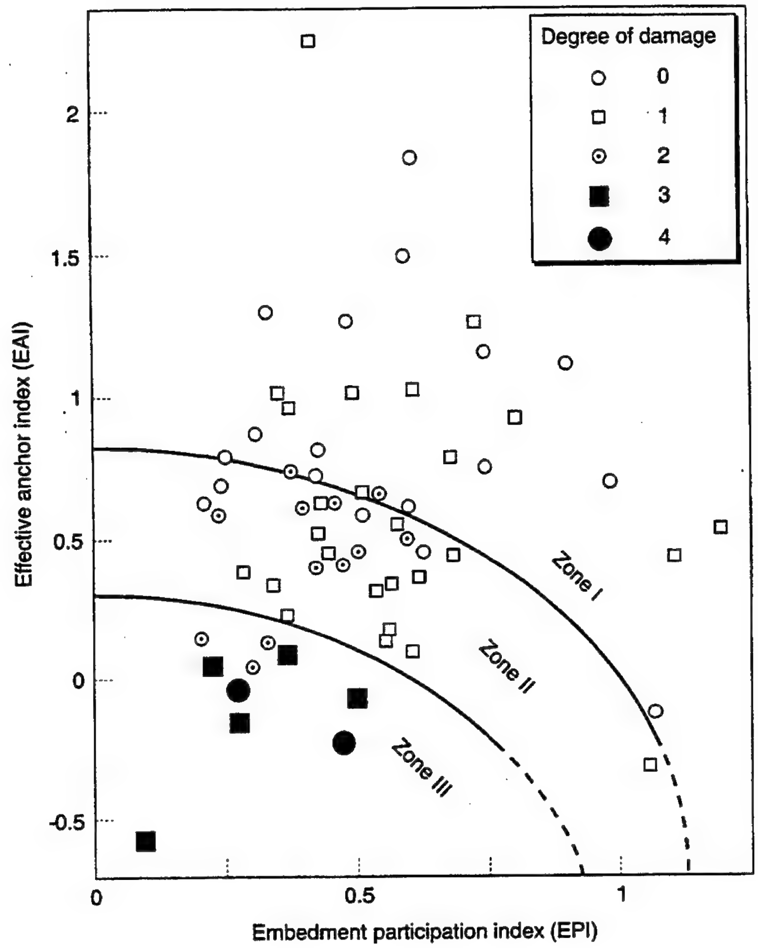

Figure 2-9: The Seismic Design Chart of Gazetas et. al. (1990).

The procedure developed by Gazetas and his coworkers (Dennehy, 1985; Gazetas et. al., 1990) establishes a minimum anchorage length for safe performance based on field observations of damaged structures at sites where surface evidence of liquefaction was lacking. In a recent study, the results of recent parametric studies (McCullough and Dickenson, 1998) which included non-liquefiable soils were compared to the design chart in Figure 2-9.. Figure 2-10 presents the comparison between the parametric study (for non-liquefiable soils) plotted as solid stars (with the calculated displacements in parenthesis) and the design chart presented by Gazetas and others. One point (center-right) is a standard 7.5 m wall. Two points (top-right and bottom-right) are from the parametric study varying the length of the tie rod anchor, and the last point (left-center) comes from the parametric study varying the depth of embedment.

It is evident from Figure 2-10 that the computed deformations vary significantly at each data point, and that some of the plotted points have calculated displacements that both fit, and do not fit the proposed design chart (especially the point on the center-right). The variations in the displacement values for each of the plotted parametric study points can be attributed to variations in the earthquake motions. Larger earthquake motions produced larger displacements, whereas the method proposed by Dennehy and Gazetas does not directly include any earthquake motion parameters (intensity, frequency or duration). It is significant to note that many of the computed deformations that fall in Zone I (deformations approximately less than 10 cm) would be considered unacceptable by many port engineers for an operating or contingency level earthquake motion (Ferritto, 1997).

A comparison can also be made between the parametric study on the depth of sheet pile embedment and the proposed chart by Dennehy and Gazetas. It was noted from the parametric study that the depth of embedment had very little effect on the performance of the bulkhead over the range of the modeled values, but the contour lines constructed by Dennehy and Gazetas show a clear variation in performance over the range of interest (EPI = 0.25 to 0.75).

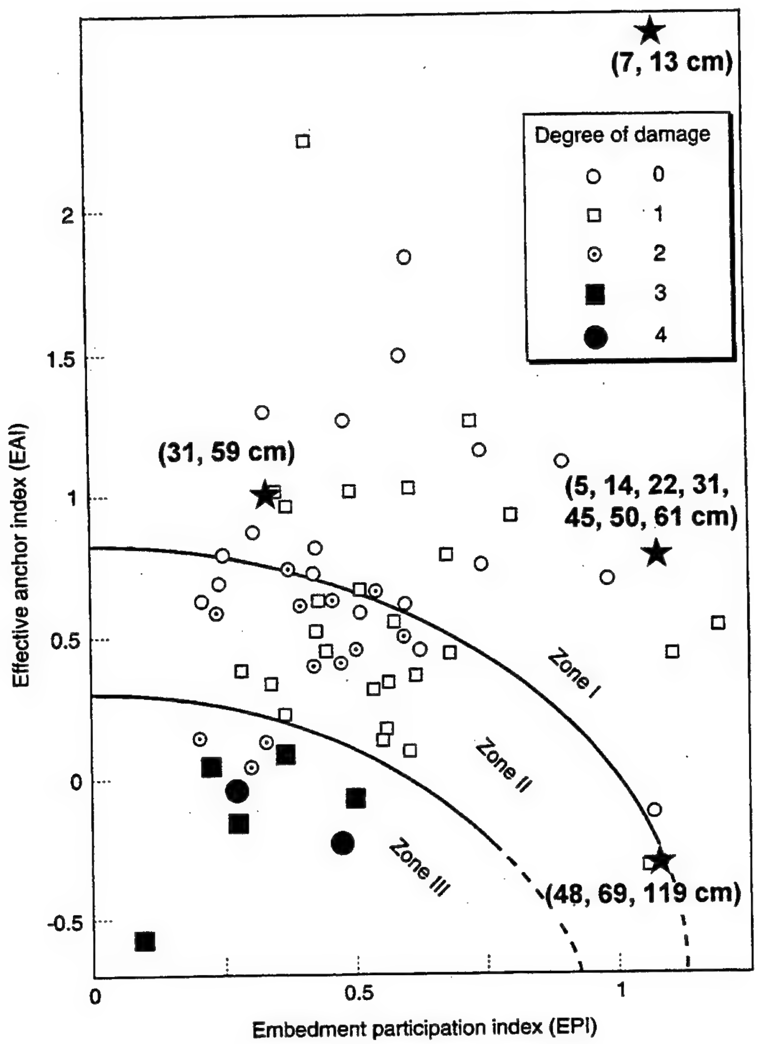

Figure 2-10: Including Data from the Parametric Study for Models with Improved Soils (solid stars)

As previously discussed for gravity retaining structures, advanced numerical modeling techniques are recommended for estimating earthquake-induced permanent displacements in displacement-sensitive sheet pile bulkheads. The applicability and benefits of advanced numerical procedures seismic analysis of sheet pile bulkheads are the same as previously discussed for gravity retaining structures.

An example of an extensive parametric study of anchored sheetpile bulkheads using a dynamic effective stress numerical model is provided by McCullough and Dickenson (1998). The evaluation of five design parameters were examined in the parametric runs, including: (a) depth of embedment of the sheetpile wall (*D*), (b) stiffness of the sheetpile wall (*EI*), (c) length of the tie rod, (d) density of the backfill soil, and (e) extent of soil

improvement (*SI*). A definition sketch of the modeled geometry is provided in Figure 2-11.

To increase the applicability of this study, normalized dimensionless factors were developed. A normalized displacement factor (Equation 3) was developed by normalizing the displacements at the top of the wall (ΔX) by the wall stiffness (EI),

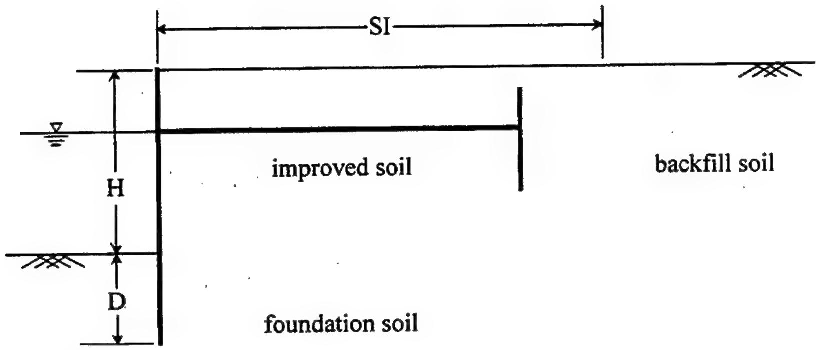

Figure 2-11: Definition Sketch Of An Anchored Sheetpile Bulkhead With Soil improvement

total wall height ($H+D$), and the buoyant unit weight of the backland soil adjacent to the sheet pile wall.

$$\frac{\Delta X \cdot EI}{(H+D)^5 \cdot \gamma_b} \qquad \text{2-8}$$

A normalized soil improvement factor (n) was also developed by dividing the extent of soil improvement (SI) by the total wall height ($H+D$).

The normalized earthquake intensity was developed by normalizing the maximum backland acceleration at the elevation of the dredge line ($A_{max@dredge}$) the magnitude scaling factor (*MSF*) (Arango, 1996). The *MSF* factors are listed in Table 2-2. In the absence of a site specific seismic study, it is recommended that the reduction factor (r_d) from Seed and Idriss (1982) be used to approximate $A_{max@dredge}$ from the maximum ground surface acceleration. The values of r_d for 15 and 7.5 meter walls are approximately 0.78 and 0.95, respectively. It should be noted that this is a one-dimensional approximation for the two-dimensional soil-structure interaction.

Table 2-2: Magnitude Scaling Factors (Arango, 1996)

Magnitude					
5.50	***6.00***	***7.00***	***7.50***	***8.00***	***8.25***
3.00	2.00	1.25	1.00	0.75	0.63

The results of the parametric study are presented in Figure 2-12. The contour lines indicate various levels of earthquake intensity for backfill soils with blowcounts of 10 and 20 blows/30 cm. The effectiveness of soil improvement for minimizing bulkhead deformations is clearly demonstrated by the design chart. It is also noted that incremental benefit of soil improvement beyond *n* values of approximately 2.0 decreases considerably. In comparison, the *n* values as determined from the PHRI (1997) recommendations for the parametric study sheetpile bulkheads are approximately 1.9 to 2.5.

There are fourteen case histories plotted on the chart, which are arranged according to the blowcounts of the backfill soils. Table 2-3 presents pertinent data from the case histories. It should be noted that seven case histories are closely predicted by the chart, five case histories are significantly over-predicted and only two of the case histories are significantly under-predicted. These results indicate that the design chart can be conservatively used as a preliminary design chart or screening tool.

The results of the study indicate that it would be very difficult to limit the deformations to 10 cm utilizing only densification methods of soil improvement for moderate to high earthquake motions ($A_{max} \geq 0.3g$). In cases such as these, it may be necessary to consider soil-cement techniques and/or structural improvements. The results of this study have been synthesized into a simplified design chart for use in estimating permanent lateral displacements for sheetpile bulkheads with or without soil improvement. This design chart is applicable for the preliminary design of new bulkheads and as a screening tool for existing bulkheads.

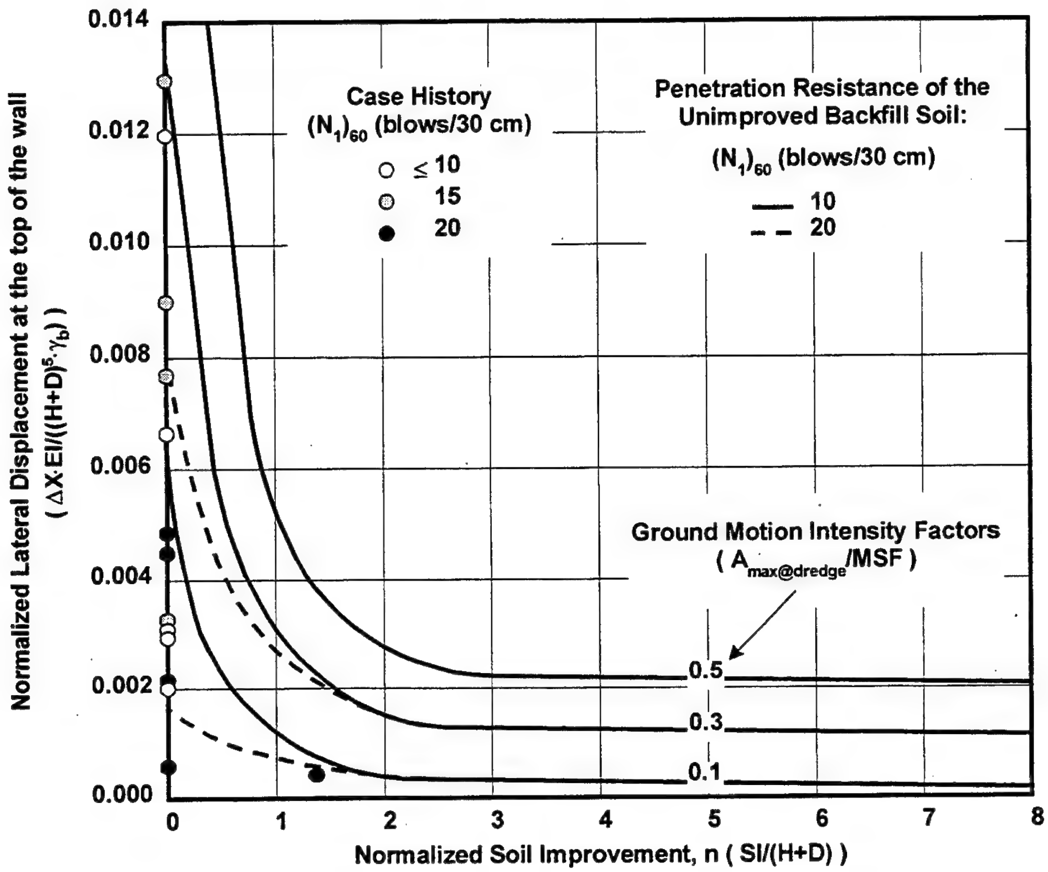

Figure 2-12: Permanent Horizontal Displacements at the Top of Anchored Bulkheads

Table 2-3: Plotted Case Histories

Earthquake	*$(N_1)_{60}$*	*$A_{max@dredge}$/ MSF (g)*	*Displacement (cm)*	*Normalized Displacement $\Delta X \cdot EI/((H+D)^5 \cdot \gamma_b)$*
1968 Tokachi-Oki	6	0.26	12 to 23	0.0120
1993 Kushiro-Oki	6	0.20	19	0.0020
1964 Niigita	10	0.14	100	0.0031
1983 Nihonkai-Chubu	10	0.10	110 to 160	0.0066
1968 Tokachi-Oki	15	0.23	16 to 57	0.0090
1968 Tokachi-Oki	15	0.23	12 to 19	0.0029
1968 Tokachi-Oki	15	0.12	30	0.0077
1973 Nemuro-Hanto	15	0.16	30	0.0031
1978 Miyagi-Ken-Oki	15	0.10	87 to 116	0.0129
1968 Tokachi-Oki	20	0.26	~60	0.0049
1983 Nihonkai-Chubu	20	0.09	~5	0.0006
1993 Kushiro-Oki	20	0.16	50 to 70	0.0044
1993 Guam	20	0.18	61	0.0019
1993 Kushiro-Oki	30	0.21	no damage (~5)	0.0004

The recommended procedures for utilizing the results of the parametric study to estimate the permanent displacement at the top of sheetpile walls, include;

1) Design the wall using current pseudo-static methods to determine the wall geometry (*H*, *D*, *EI*, and anchor length).

3) Determine $A_{max@dredge}$ from a site-specific seismic study or an approximate empirical relationship.

4) Determine the earthquake intensity factor for the specific earthquake by dividing the magnitude scaling factor into $A_{max@dredge}$.

5) Based on the density of the backfill and the extent of soil improvement, estimate the permanent lateral displacement at the top of the sheetpile wall (ΔX) using Figure 2-12 and the normalized displacement equation (Equation 2-8).

Current pseudo-static design methods allow for determination of the sheetpile wall section, tie rod length, and depth of embedment, but since they are limit-equilibrium based, it is not possible to estimate lateral deformations using these methods.

Guidelines for Seismic Design of New Components

As previously mentioned regarding the seismic design of gravity retaining structures, the seismic design of steel sheet pile bulkheads must focus on the dynamic behavior of the foundation and backfill soils, as well as the overall stability of the bulkhead wall retaining structure. Widely used limit equilibrium methods require that potential seismically-induced movement of the walls be estimated in order to evaluate the state of stress in backfill soils (i.e., yielding versus non-yielding backfills). The flexibility of anchored sheet pile bulkheads has led designers to assume yielding backfill and employ the dynamic earth pressure method of Mononobe and Okabe (as outlined by Ebeling and Morrison, 1993). Enhancements to the standard pseudostatic methods of analysis have been made by numerous investigators (e.g., Neelakantan et. al., 1992; Power et. al., 1986; Steedman and Zeng, 1990). It is noted that these methods do not indicate the lateral deformations of the bulkhead that would likely occur during the design earthquake.

Based on an extensive review of seismic performance data for anchored sheet pile bulkheads, Kitijima and Uwabe (1979) concluded that the level of damage to these structures is related to the permanent deformations of the top of the bulkhead wall during the earthquake. Their general observations are summarized in Table 2-4, and underscore the importance of the structure deformations and the associated lateral ground movement for the establishment of seismic design criteria for anchored sheet pile bulkheads. Rigid body, "sliding-block" type displacement analyses have been used as the basis for estimating the seismically-induced movement of anchored retaining structures (Towata and Islam, 1987). In addition, a semi-empirical method based on the performance of

anchored bulkheads at sites which did not exhibit significant liquefaction has been developed by Gazetas et. al. (1990) for estimating the deformations of anchored bulkheads based on dynamic earth pressures and the bulkhead-to-anchor spacing. These techniques are recommended as initial screening methods for evaluating the seismic performance of anchored bulkheads.

Table 2-4
Relationship Between The Deformation Of Anchored Sheet Pile Retaining Walls And Observed Damage
(Kitijima And Uwabe, 1979)

Description of Damage	Permanent Displacement at Top of Sheet Pile	
	cm	inches
No Damage	<2	<1
Negligible Damage to Wall itself, and Noticeable Damage to Appurtenant Structures	10	4
Noticeable Damage to Wall	30	12
General Shape of Anchored Sheet Pile Preserved, but Significantly Damaged	60	24
Complete Destruction. No Recognizable Shape of Wall	120	48

As with all marginal wharf structures, seismic hazards associated with soil liquefaction must be mitigated in order to reduce earthquake-induced deformations to within allowable limits. Soil improvement techniques have been used at ports throughout the world to increase the liquefaction resistance of soils adjacent to waterfront retaining structures. Although ground treatment is applicable adjacent to anchored sheet pile bulkheads, the flexible nature of the bulkhead walls is such that lateral deformations should be expected even in competent, non-liquefiable soils subjected to high intensity

shaking. As mentioned in previous sections of this chapter, the allowable deformations will reflect the sensitivity of appurtenant structures. It should also be noted that several of the soil improvement techniques may not be applicable in close proximity to the sheet pile bulkhead (within 20 ft. or less). In several documented case studies, lateral deformations of the backfill soils adjacent to the bulkhead during densification or grouting have resulted in increased loads in the tie rods and wales, and increased bending stresses in the piles (PHRI, 1997). Finally, structural measures may be required to restrengthen anchor systems. Retrofit strategies could include a second row of anchors tied to the first, the construction of more robust anchors (i.e., larger deadman, larger piles, etc.), reconstructed tie rod-wale connections.

The advantages and disadvantages of the various analytical procedures for analyzing the seismic performance of sheet pile structures (e.g., pseudostatic limit equilibrium, sliding block type analyses, advanced numerical modeling) limit are similar to those outlined for gravity retaining structures.

Guidelines for Seismic Retrofit of Existing Components

Experience during past earthquakes demonstrates that liquefaction-related phenomena constitute the primary seismic hazard to sheet pile bulkheads. Therefore, one of the most effective measures that can be used to reduce the seismic risk due to failure of such structures is the use of soil improvement adjacent to the bulkhead wall and anchor system (e.g., vibro-techniques, stone columns, gravel drains, grouting). Soil improvement may also be required in front of wall to ensure adequate passive resistance below the dredge line. Guidelines for the utilization of soil improvement for sheet pile bulkheads have been presented PHRI (1997). Figures 2-13 and 2-14 show their recommended layout for the volume and extent of soil improvement that is required to minimize the bulkhead deformations to allowable levels under various levels of ground shaking. Again, as previously noted, design engineers should keep in mind that several of the soil improvement techniques may not be applicable in close proximity to the bulkhead. Lateral deformations of the backfill soils adjacent to the bulkhead during densification or grouting have been observed to increase loads in the tie rods and wales.

Re-evaluation of the bulkheads may indicate that anchor systems should be further strengthened. In these situations, retrofit strategies could include a second row of anchors tied to the first, and the construction of more robust anchors (i.e., larger deadman, larger piles, etc.), reconstructed tie-rod\wale connections.

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CHAPTER 3 STRUCTURAL CRITERIA FOR PIERS AND WHARVES

Introduction

Piles are a common element to much of the waterfront construction and recent experience has shown them vulnerable to damage. It is very difficult to inspect and repair damage which occurs to piles underground or under water; therefore, it is desirable to design piles to limit damage under the range of possible earthquakes. For the design earthquake, Level 1, which is expected to occur one or more times during the life of the structure, the piles should be undamaged. For the upper bound earthquake, Level 2, which is a rare event, the structure must sustain limited controlled damage; under such conditions it is desirable that the seismic energy be dissipated by ductile yielding at plastic hinge regions.

General Waterfront Damage Mechanisms

Werner and Hung (1982) gives an excellent compilation of case studies mostly recounting Japanese experiences from the 1920's to 1980. They conclude that "By far the most significant source of earthquake-induced damage to port and harbor facilities has been porewater pressure buildup... which has led to excessive lateral pressures applied to quay walls and bulkheads." They cite the 1964 Niigata and 1964 Alaska earthquake where "porewater pressures buildup has resulted in complete destruction of entire port and harbor areas" They note that direct effects of earthquake induced vibrations on waterfront structures is minimal and overshadowed by liquefaction induced damage. Failure of bulkhead anchorage systems is a common significant damage inducing mechanism. Liquefaction also causes damage to piles. The Anchorage City Dock was a reinforced concrete structure supported on pipe pile with diameters from 16 to 42 inches. Some of the piles were batter piles and filled with concrete. The piles were supported on clay which consolidated and settled 4 feet. This movement resulted in deck displacements from 8 to 17 inches buckling the batter piles (Tudor/PMP, 1976) Experience from Niigata and Alaska suggests that piles deform with the soil. In the 1970 Peru earthquake, magnitude 7.8, the Sogesa Wharf suffered severe damage when the inboard piles restrained by the dike structure could not tolerate high displacements, Tudor/PMP (1976).

Table 3-1 from Werner and Hung (1982) and updated in Werner (1998) gives case studies. The first paper warns of the vulnerability of batter piles as would be observed seven years later in the Port of Oakland. Gazetas and Dakoulas (1991) evaluate numerous waterfront case histories including the performance of sheetpile bulkheads in which the major failures have resulted from large-scale liquefaction in the backfill or supporting base. Frequently the anchored bulkhead damage takes the form of excessive outward movement and tilt caused by excessive movement of the anchor. They show that Japanese code procedures were inadequate because the accelerations values are often

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 1 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude	Location	PGA (g)[1]	Description	Cause(s)[2]
Kanto, Japan	09-01-23	8.3 (M_w)	Yokohama and Yokosuka	-	Concrete Block Quay Walls: Sliding, tilting, and collapse with some bearing capacity failure of rubble foundation. Steel Bridge Pier: Buckling of pile supports.	A C, E
Kitaizu, Japan	11-26-30	6.9 (M_w)	Shimizu	-	Caisson Quay Wall (183 m long): Tilting, outward sliding (8.3 m), and settlement (1.6 m). L-Shaped Concrete Block Quay Wall (750 m long): Outward sliding (4.5 m).	A, B, C
Shizouka, Japan	07-11-35	6.2 (M_w)	Shimizu	-	Caisson Quay Wall: Outward sliding (5.5 m), settlement (0.9 m), and anchor system failure.	A, B, C
Tonoakai, Japan	12-07-44	8.1 (M_w)	Yokkaichi Nagoya Osaka	0.30-0.35	Pile-Supported Concrete Girder and Deck: Outward sliding of structure and soil (3.7 m). Sheet-Pile Bulkhead with Platform: Outward sliding (3.7 m). Steel Sheet-Pile Bulkhead: Outward bulging (3 m).	A, B, C
Nankai, Japan	12-21-46	8.3 (M_w)	Nogoya Yokkaichi Osaka Uno	0.25-0.30	Steel Sheet-Pile Bulkhead with Platform: Outward bulging (4 m). Pile-Supported Concrete Girder and Deck: Outward sliding (3.7 m). Steel Sheet-Pile Bulkhead: Outward bulging (3 m) and settlement. Gravity-type Concrete-Block and Caisson Quay Wall: Seaward sliding (0.4 m).	A, B, C

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1

SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 2 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude	Location	PGA (g)[1]	Description	Cause(s)[2]
Olympia, Washington, USA	04-13-49	7.1 (M_s)	Seattle	-	Dock Structures: Major damage to structures and to goods awaiting shipment. Loaded tanker tied to dock reported vertical displacement of about 1 inch at a frequency of about 3 Hz. Soil Materials: Stuctures in vicinity of waterfront located on filled ground suffered significant damage. Major water spouts observed from ground fissues.	B
Tokachi-Oki, Japan	03-04-52	8.1 (M_w)	Kushiro	-	Concrete Caisson Quay Wall: Tilting, outward sliding (6 m), and settlement (1 m).	A, B, C
Chile	05-22-60	9.5 (M_w)	Puerto Montt	-	Concrete Caisson Quay Walls: Overturning and extensive tilting. Steel Sheet-Pile Sea Wall: Outward sliding (up to 1 m) and anchor failure. Gravity-Type Concrete Sea Wall: Complete overturning and sliding (1.5 m).	A, B, C
			Talcuhauno	-	Concrete-Block Quay Wall: Outward tilting.	A, B
Alaska, USA	03-27-64	9.2 (M_w)	Anchorage	-	Dock Structures: Extensive seaward tilting with bowing, buckling, and yielding of pile supports.	B, D, E
			Valdez		Entire Port: Destroyed by massive submarine landslide.	B, D, F
			Whittier		Pile-Supported Piers and Docks: Buckling, bending, and twisting of steel pile supports.	B
			Seward		Major Portion of Port: Destroyed by massive submarine landslide.	F
Niigata, Japan	06-16-64	7.6 (M_w)	Niigata	0.15-0.20	Caisson Quay Wall (183 m long): Tilting, outward sliding (8.3 m), and settlement (1.6 m). L-Shaped Block Quay Wall (750 m long): Outward sliding (4.5 m).	A, B, C

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 3 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude.	Location	PGA (g)[1]	Description	Cause(s)[2]
Puget Sound, Washington USA	04-29-65	6.5 (M_s)	Seattle	-	General: Nearly every waterfront facility damaged to some extent. Piers 15 and 16 on Harbor Island: Structures shifted toward water by about 1 foot due to the soil losing strength or partially liquefying and pushing dock toward water Pier 5: Among the hardest hit pier areas. Bulkhead and fill behind it settled as much as 2 ft, over width of 25-40 ft. Bulkhead reported to be 6-8 in. out of line. Buildings and Utilities: At Pier 36, light fixtures were ton loose in 5-story concrete Engineers' Headquarters Building. File cabinets tipped over and most books in library fell from shelves. Damage reported to Pier 90 waterline and Pier 91 steamline.	A, B, E
Tokachi-Oki, Japan	05-16-68	8.3 (M_W)	Hachonohe	0.20-0.25	Steel Sheet-Pile Bulkheads: Outward sliding (0.9 m), tilting, and settlement, with anchor failure.	A
			Aomori	0.20-0.25	Gravity-type quay wall: Sliding and settlement (0.4 m). Gravity-type breakwater: Sliding (0.9 m) and pavement settlement (0.9 m).	A
			Hakodate	0.10-0.15	Steel sheet-pile bulkhead: Seaward tilting (0.6 m) and apron settlement (0.3 m). Quay wall: Settlement (0.6 m) and sliding (0.4 m).	A, B
Nemuro-Hanto-Oki, Japan	06-17-73	7.9 (M_W)	Hanasaki	0.25-0.30	Gravity-Type Quay Wall: Sliding (1.2 m), wall settlement (0.3 m), and apron settlement (1.2 m). Steel4-22 Sheet-Pile Bulkhead: Sliding (2 m) and anchor failure.	A, B
			Kiritappu	-	Steel Sheet-Pile Bulkhead: Relatively minor damage. Gravity-Type Quay Wall: Relatively minor damage.	

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 4 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude.	Location	PGA (g)[1]	Description	Cause(s)[2]
Miyagiken-Oki, Japan	06-12-78	7.6 (M_W)	Shiogama	0.25-0.30	Concrete Gravity-Type Quay Wall: Outward tilting (0.6 m) and apron pavement settlement (0.4 m).	A, B
			Ishinomaki	-	Steel Sheet-Pile Bulkhead: Outward sliding (up to 1.2 m) and apron settlement (up to 1 m). Concrete-Block Retaining Wall: Sliding, tilting, and cracking with corresponding pavement settlement (0.2 m) relative to wall.	
			Yuriage	-	Concrete-Block Gravity Quay Wall and Steel Sheet-Pile Bulkhead: Horizontal displacement (1.2 m).	
			Sendai	-	Steel Sheet-Pile Bulkheads: Cracking and settlement of apron and pavements.	
					Seawall at Passenger Terminal: Lateral movement of seawall, settlement of backfill.	
Offshore Central Chile	03-03-85	7.9 (M_W)	San Antonio	-	Concrete-Block Seawall at Western Extremity of Port: Collapse of about 60 percent of seawall length, leading to significant tilting of crane structures.	A, B
			Valparaiso	-	Seawalls at Sites 6 and 7 (West Side of Port): Several inches of soil movement at top of wall, causing movement of crane rails and disruption of crane operations.	
Kalamatz, Greece	09-13-86	6.0 (M_W)	Kalamatz	0.15-0.27	Harbor Area: Soil conditions in Kalamata reportedly consist of non-uniform sandy layers intermixed with coarse sand and gravel or with varying proportions of silt and clay. No evidence of pore pressure buildup or liquefaction was observed. Very minor damage in harbor area reportedly consisted of some slope movement at wharf and along nearby coastal road, causing some pavement cracking.	C

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 5 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude.	Location	PGA (g)[1]	Description	Cause(s)[2]
Loma Prieta, California, USA	10-17-89	6.9 (M_W)	Redwood City	0.22-0.28	Waterfront Components: Damaged batter piles, separation of concrete ramp from pier at Wharf 1. Infrastructure Components: Broken water lines, temporary disruption of power and communications. Port Operations: Not disrupted.	A, B
			Richmond	0.10-0.15	UNOCAL Terminal: Ruptured gasoline storage tank. Ford Plant east of Terminal 3: Broken water lines. Port Operations: Port fully functional two days after earthquake.	A, B
			San Francisco	0.15-0.20	Piers Supported by Fills (Rather than by Piles): Settlement. Piers Supported by Piles: Many broken batter piles. Cracked concrete decks above piers. Buildings along Waterfront: Significant structural damage (including clock tower at Ferry Building). Container Cranes: Damaged. Infrastructure Components: Many broken water lines. Pier 80 Terminal: This 1950s vintage wharf on batter-pile system was upgraded in mid-1980s to support larger 100-foot gage crane rail. New vertical pile supports for this crane rail doubled as new lateral seismic force resisting system for entire wharf. Wharf undamaged during earthquake.	A, B, E
			Oakland	0.25-0.30	Seventh Street Terminal: Sand fill dike under landside crane rail settled up to 18 in. and moved laterally about 6 in. Settlement of crane rail, broken batter piles, separation of piles from deck. South perimeter dike had one location where settlements exceeded 3 ft. and lateral spread displacements were about 2-4 ft. Extended closure of terminal. Matson Terminal: Separation between piles and deck slabs. Cracking of piles. Yard settlements of up to 2 ft. Terminal stayed in operation but with limited live loads during crane operations. Middle Harbor Terminal: 0.5 to 2 ft. of subsidence in yard areas. Howard Terminal: Some lateral movement of landslide crane rail (the crane remained operational). Good performance of vertical piles designed as ductile moment frame system. Marine Operations Building located just behind dike settled about 12 in., with no damage other than utility connections. This was small two-story building on flat slab that floated on liquefied soils. Liquefaction: In yards of each of above terminals.	A, B, C

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1

SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 6 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude	Location	PGA (g)[1]	Description	Cause(s)[2]
Philippine Islands	07-16-90	7.7 (M_W)	San Fernando	-	Government Pier No. 1: Significant cracking, buckling, and deformation of reinforced concrete pile-supported pier. Extensive cracking of concrete piles, and particularly severe damage to batter piles. Sever cracking of adjacent concrete sea wall. Operations suspended. Coal Pier: Large lateral movement caused concrete slab between pier and sea wall to fall off its seat. Dry Bulk Handling Equipment: Collapsed.	A, B, C
Costa Rica	04-22-91	7.4 (M_W)	Limon	-	Soil Conditions: Founded on fill beyond natural shoreline and an old breakwater. North Dock Area: Severe liquefaction, with settlement and lateral spreading of concrete slabs, abutments, and retaining walls. Displacements of up to ½ meter. Intense east-west shaking caused major damage to large 2-story warehouse, including failure of second floor and collapse of middle one-third of building (resulting in serious injuries to five people). Permanent deformations at dock expansion joints hampered post-earthquake crane operations.	A, B, E
			Moin	-	Soil Conditions: Relatively stable soils, with no obvious signs of liquefaction-induced settlement. Port Operations: Normal operations restored soon after earthquake. Coastal uplift (of up to 1 meter) caused fire pump suction lines mounted on pier to terminate about 0.1 meter above water level..	
			Almirante (Panama)	-	Railroad Trestle: Concrete piled bents severely damaged, closing trestle to rail traffic. Concrete Pier: Severe damage, resulting in significant reduction of ship-loading capacity.	B, E
Hokkaido Nansei-Oki, Japan	07-12-93	7.8 (M_W)	Adane, Esashi	-	Widespread damage due to tsumami damage and scour. Structural damage to buildings due to differential ground settlement.	B, F
			Hakodate and Setana	-	Quay walls displaced or tilted by up to 1 meter. Damage to paved aprons due to ground settlement.	B
			Okushiri-Cho	-	Fuel lines damaged by ground settlement and permanent lateral ground displacement. Ferry building damaged by tsunami impact and ground settlement. Portions of concrete breakwater toppled by tsunami and lateral spreading of foundation soils. Fuel tank buckled due to landslide.	B B, C, F

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 7 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude	Location	PGA (g)[1]	Description	Cause(s)[2]
Guam	08-08-93	7.7 (M_w)	Apra Harbor	0.20-0.30	Navy Facilities: Damage to piers founded in dredged backfill. Sheet-pile bulkheads and anchor systems moved as much as 3 ft. Damage to batter piles near pile caps. Commercial Facilities: Lateral spreading in container-handling portion of port.	A, B
Northridge, California, USA	01-17-94	6.7 (M_w)	Los Angeles	0.15-0.20	Berths 121-126 of American President Lines Terminal: Seaward movement of berths (0.5 ft.) and settlement and liquefaction of backland fill areas. Dikes settled. Damage to 171-lb crane rail. Several days of down time before full operations restored. Newer portion of wharfs (with vertical piles designed as ductile moment frame and with 86 ft. rock dike) performed well. Some damage to vertical and batter piles at older portions of Berth 126. Other Terminals. Settlement, lateral spreading, liquefaction of fills. Some older facilities damaged. Infrastructure Components. Ruptured water lines, loss of power to some cranes (which interrupted crane operations), and interruption of telephone service (for up to several hours).	A, B
			Long Beach	0.10-0.15	No damage reported.	
Hyogoken Nanbu, Japan	01-17-95	6.8 (M_s)	Kobe	0.20-0.70	International Shipping: Virtually all container terminals on Port Island and Rokko Island severely damaged and inoperable, due to liquefaction of fills along waterfront areas. Seaward movement of caisson quay walls (up to 25 ft.) and associated vertical settlement (up to 10 ft.) of fills behind quay walls. Large lateral deformations extended into backland areas over distances of 250-350 ft. Cranes suffered, de-railing, overturning, and buckling of legs. Domestic Shipping: Major damage to ferry terminals and to main trunk line for domestic commodity distribution. Quay walls on piles performed much better than caisson quay walls (although damage to lightly reinforced concrete piles and buckling of steel pipe piles was observed). Infrastructure. Good performance of symmetrical shear wall buildings. Poor performance of buildings with soft stories or other lateral load resistance deficiencies. . Reduced port access due to damage to highway transportation system. Extensive damage to water lines.	A, B, E
			Osaka	0.20-0.27	General: Good overall seismic performance, due to such factors as the use of clayey fill thoroughout the port, and densification of sandy trench fill beneath caissons at North Port.	

Notes: (1) PGA = Peak ground acceleration (estimated or recorded); (2) Legend for causes of earthquake damage given on Page 8 of table.

Table 3-1
SUMMARY OF EARTHQUAKE-INDUCED DAMAGE TO PORT FACILITIES (Page 8 of 8)

Earthquake			Port		Damage	
Location	Date	Magnitude	Location	PGA (g)[1]	Description	Cause(s)[2]
Antofagasta, Chile	07-30-95	7.8 (M_s)	Antofagasta	-	Port of Antofagasta is forth largest port in Chile, and is most important port for copper exports in northern section of country. Settlements of up to 0.6 meter resulted in reduction of cargo handling capability of port to only 20 percent of its pre-earthquake capability. Almost all cranes in port suffered operational damage. Almost no damage to warehoused founded on piles.	B
Manzanillo, Mexico	10-09-95	7.9 (M_s)	Manzanillo	0.40	Some liquefaction with considerable differential settlement in areas of this industrial port located on dredged fill overlying reclaimed estuary wetlands. Significant lateral spreading of dredged fills where not contained by rock dikes. Two large gantry cranes at new container facility for off-loading container ships had both crane rails located on pile-supported wharf, and were not damaged during earthquake. However, crane runway rail shifted laterally causing offsets at rail splice. For container cranes to gantry entire length of wharf, sections of these rails had to be readjusted. Moderate damage to longitudinal and transverse batter piles, and concrete spalling of about 10% of underside of deck. About 80 batter piles either completely severed or suffered extensive damage. Port buildings that were either not pile-supported or only partially pile-supported in areas with differential settlement suffered architectural and structural damage. Buildings supported totally on piles performed well. Some water and sewage lines ruptured.	B,C
Gulf of Aqaba, Egypt, Jordan, and Israel	11-22-95	7.2 (M_s)	Eilat, Israel	0.08-0.11	General: Relatively light damage. Some liquefaction and sand boils in port area. Ground fracture and cracking of road surfaces. Some cracking of infill walls and at building expansion joints.	B
			Nuweiba, Egypt	0.16-0.25	Waterfront Facilities: Two out of four berths were severely damaged, due to liquefaction of fill materials. Significant seaward rotation of quay walls, leading to separation between capping beam and backfill. Excessive settlement of apron slabs. Other Port Facilities: Collapse of stone walls, and architectural masonry walls of arrival hall. At desalination plant, sliding of unanchored steel tank and buckling of tank shell. Sliding of transformer at electrical substation damaged conduits and caused loss of power supply.	A, B

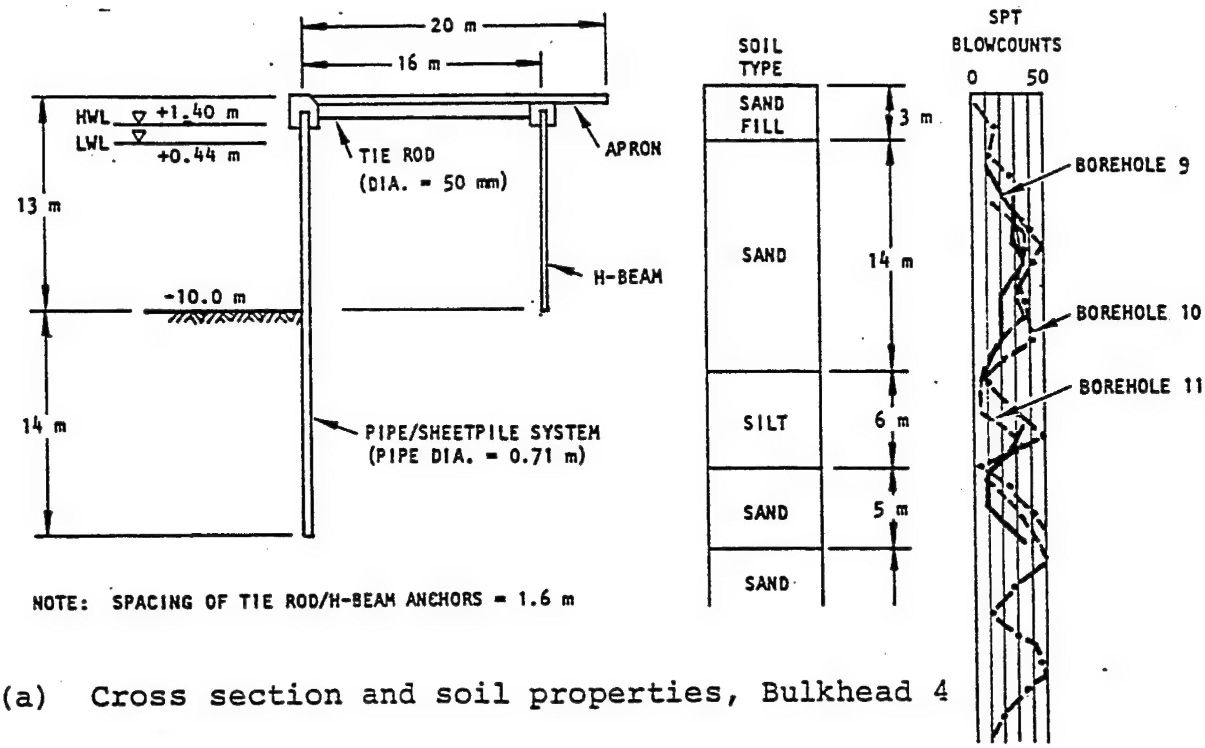
Notes:
(1) PGA = Peak ground acceleration (estimated or recorded).
(2) Legend for causes of earthquake damage is as follows:
A: Large lateral pressure from backfills. B: Liquefaction. C: Localized ground movement
D: Massive submarine sliding. E: Structural vibrations. F: Tsunami.

exceeded and the vertical component is omitted, they neglect ground motion amplification, and they do not take into account lateral soil spreading caused by a loss of stiffness. They note limitations in traditional pseudostatic sheetpile procedures to properly define the location of the active sliding surface. They develop an empirical seismic design chart based on the observed case histories. Other works of significance include Swanson (1996) which summarizes observed damage in the Kobe earthquake, Harn and Malick (1992) which gives design guidance, and Erickson and Fotinos (1995) which summarizes various code requirements.

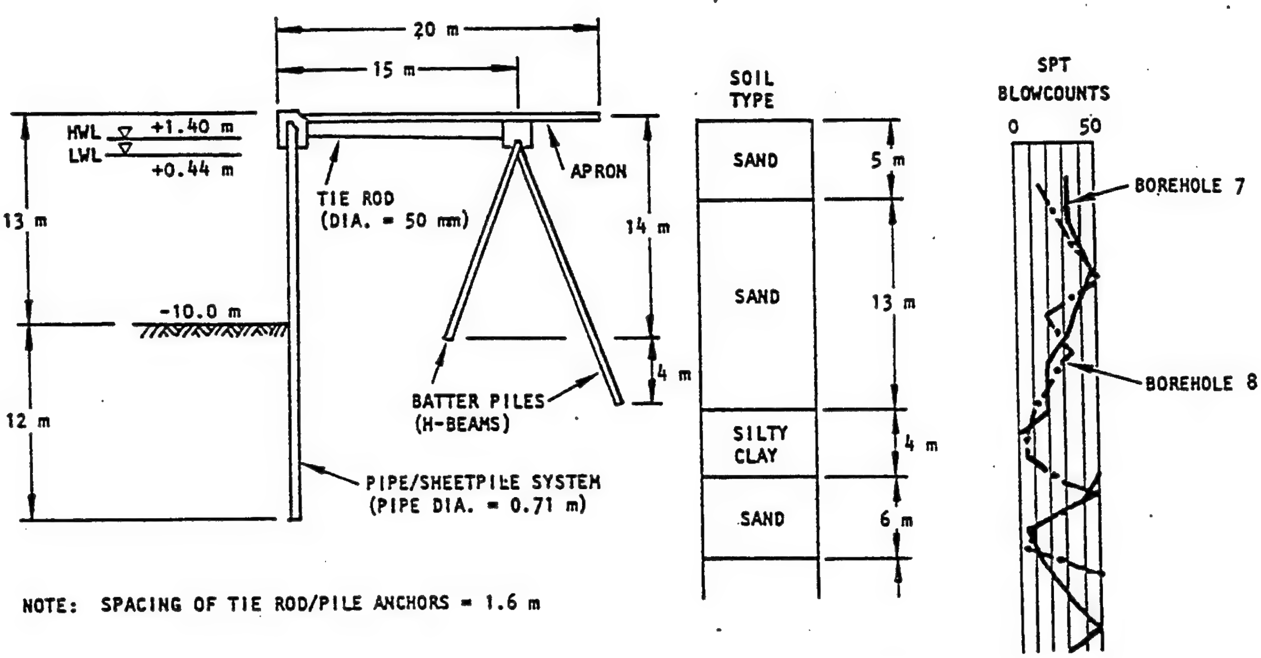
The 1978 magnitude 7.4 earthquake Miyagi-Ken-Oki earthquake caused severe damage to gravity quay walls, piers and sheet pile bulkheads. The Sendai Port area has a soil profile composed of a sand layer 3 to 20 meters thick underlain by layers of medium coarse sand and silty loam. Dense sand and bedrock underlie the silty loam layer. Two nearby bulkheads serve as a comparison study, Figure 3-1. A seismic lateral coefficient of 0.1 g was used in the design. Bulkhead No. 4 was anchored with vertical H-beam. The area behind this bulkhead experienced cracking and settlement.. Bulkhead No. 5 was constructed in a similar manner except that it used batter piles to restrain the anchor. This bulkhead withstood the earthquake without damage. Note as shown in Figure 3-1 the near surface soil behind Bulkhead No. 4 had lower blowcounts which when combined with reduced anchorage could have caused the increased lateral spread and associated damage.

Damage To Waterfront Structures Having Piles

The 1989 Loma Prieta Earthquake caused major port damage to the Port Of Oakland, Table 3-1. Soil liquefaction caused damage to the terminal facilities much of which is filled land composed of loose dumped or hydraulically placed sand underlain by soft normally consolidated Bay Mud. There were four areas damaged: the 7th Street Terminal, the Matson Terminal, the APL Terminal and the Howard Terminal. All of these terminals had pile supported wharves typically represented by Figure 3-2. The piles extended through the rock dike which served as containment for the fill composed of fine dredged sands and silty sands. The most severe damage occurred at the 7th Street Terminal where liquefaction of the fill resulted in settlements and lateral soil spreading, cracking the pavement over a wide area. Maximum settlements of the paved yard area were up to 12 inches. The inboard crane rail was supported on the fill directly which settled; the outboard crane rail was supported on the wharf piles and did not settle. As a result of this differential movement the cranes were inoperable. Damage occurred to the tops of the batter piles, Figure 3-2, through shear, compression, and tension. The vertical piles were largely undamaged with a few exceptions. The stiff batter piles absorbed much of the loading among the other more flexible elements. Seed et al. (1990) suggests "the mode of failure was predominantly tensile failure driven by outward thrust of the fill, suggesting that liquefaction and associated spreading were important factors". As a result of this damage the port of Oakland is replacing all the 7th Street Terminal batter piles with vertical piles designed to resist lateral forces. The pile-wharf deck is being extended



(a) Cross section and soil properties, Bulkhead 4



(b) Cross section and soil properties, Bulkhead 5

Figure 3-1. Bulkheads 4 and 5 at Nakano wharf, Sendai Port.

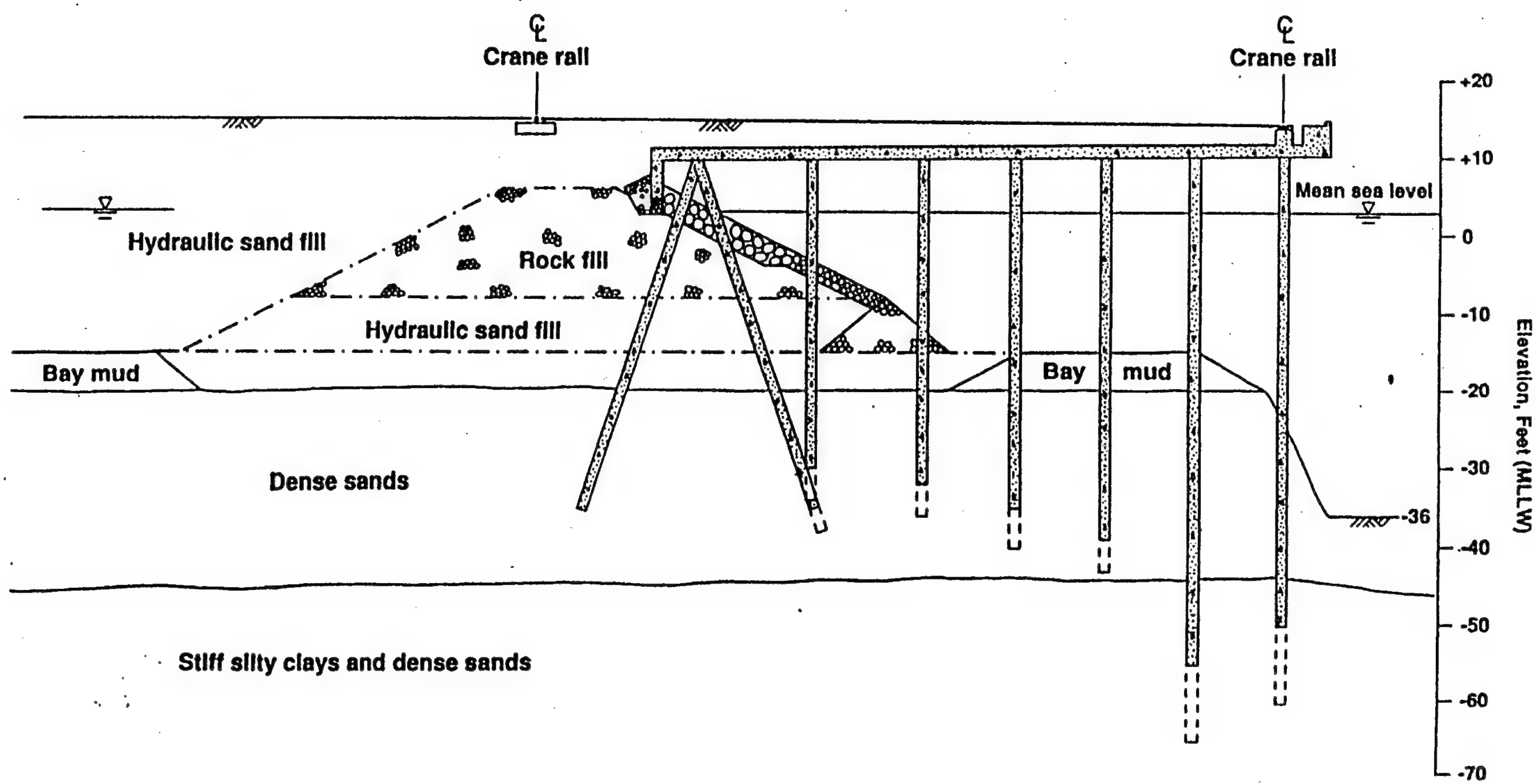


Figure 3-2. Cross-section 7th St Terminal, Port of Oakland.

inboard to provide support for the crane rails. The Howard Terminal and the APL Terminal which had vertical or near vertical piles instead of batter piles did not sustain pile damage although liquefaction caused comparable settlements in the filled areas. Both crane rails were also pile supported.

On August 8, 1993 a magnitude 8.1 earthquake occurred offshore 50 miles from Guam and caused over $125 million in damages to Naval facilities on Guam, Table 3-1. Nearly all of Guam is firm soil or rock except for the region containing the Navy port which had soft soil composed of natural alluvium and artificial fill. It is estimated the peak horizontal ground accelerations were about 0.25g. Liquefaction was a major problem and lateral spreading of 1 to 2 feet was observed at wharf areas. It also resulted in settlements, backfill collapse and bulkhead movements. Buried water and power lines were fractured. Sheet piles failed in shear and deadman anchors pulled out. Batter piles failed in shear at the pile cap. Other Navy damage consisted of fuel tank leaks, sloughing of a dam, damage to masonry housing units and major damage to the power plant which supplied 20 percent of the island's power capacity.

In January 1995, the Hyogo-ken Nambu (Great Hanshin Kobe) earthquake, Japanese magnitude 7.2 (about 6.9 moment magnitude), occurred in Kobe Japan. This event produced major damage to Japan's second busiest port. Liquefaction was a major contributor to the extent of the damage producing typical subsidence of a half meter. Piles were used extensively in this area. They were designed to account for the negative skin friction and additional ground improvement was also performed. Structures on such piles performed well even though major subsidence occurred in surrounding areas. Other structures not on piles suffered differential settlement and tilting and significant damage. Liquefaction caused up to 3 meters of lateral spread displacement, sunk quay walls, broke utility lines, and shut down 179 out of 186 berths at the port. It was responsible for major damage to crane foundations. Hydraulic fill behind concrete caisson perimeter walls fill liquefied causing the caissons to move outward rotating up to 3 degrees and settling from 0.7 to 3.0 meters. The caissons were designed for a lateral coefficient of 0.1g. A seismic coefficient of 0.2g was normally specified for dockside cranes. Peak accelerations of 0.8g in the NS direction, 0.6g in the EW direction and 0.3g vertical were noted from accelerograph recordings. The event had a duration of about 20 seconds. The outboard crane rails which were supported on the caisson also spread outward, Figure 3-3. The middle crane rails which were supported on piles did not move. The inboard crane rails settled between 1 and 2 meters. The increase in distance between crane rails resulting from lateral spreading was from 1 to 5 meters. Both old and new caisson construction faired equally poorly. The resulting deformation disabled all the dockside container cranes collapsing one and shutting down all port operations. Damage is attributed to liquefaction since structures supported on pile suffered much less damage, Liftech (1995). It should be noted that caissons designed for 0.25g sustained lower levels of damage.

Pile designs must be checked for the location of the maximum moment, generally at the pile cap. The second highest point is within the support soil. Damage below the soil line cannot be seen and easily repaired; thus, consideration of this must be included in the

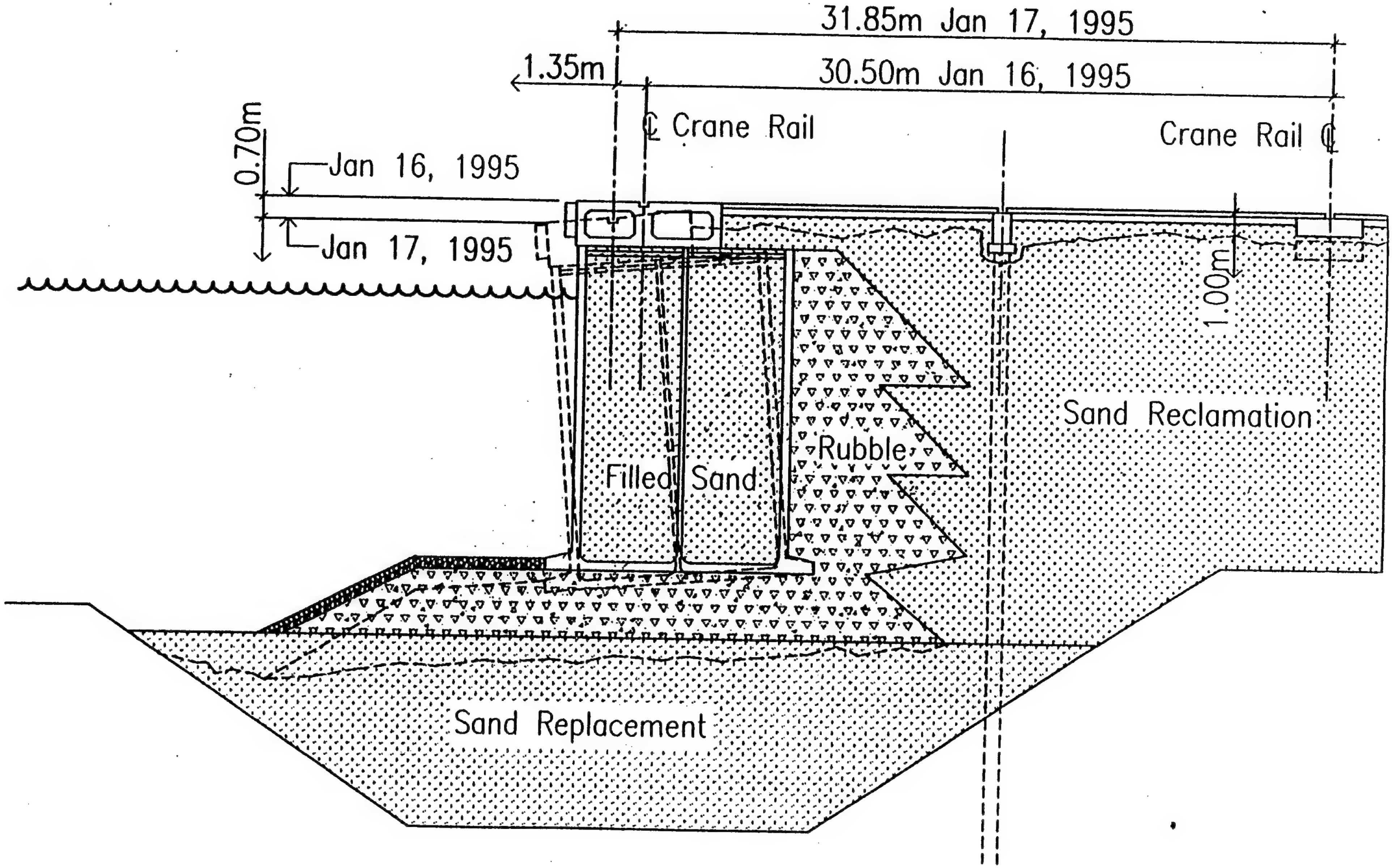


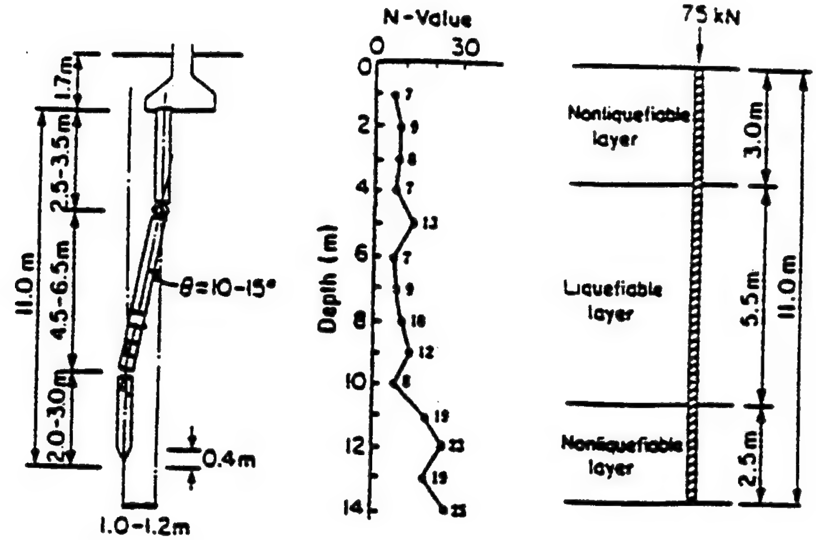
Figure 3-3. Typical wharf, Kobe Japan.

design. POLA provides for decreased ductility for wharf pile sections below the soil line. It is important to note that the pile curvatures are influenced by the stiffness of the supporting soil. Soil movement may be concentrated at interfaces between stiff and soft soils causing local increases in curvature, overstressing the pile. Soil layers and their associated stiffness must be accurately modeled in a pile analysis.

During the 1995 earthquake in Kobe Japan damage occurred to precast concrete piles on Port Island, Matso, K. (1995). Typically the failure mode consisted of anchor reinforcing bars pulling out of the pile caps producing separation between the piles and the pile cap. The Hanshin Expressway which collapsed was an elevated roadway supported on a series of single concrete pier. The failure of the pier was associated with failure of the transverse shear reinforcement and premature termination of longitudinal reinforcement. This reinforcement consisted of perimeter ties lapped at the ends and was spaced 30 cm on center. The inadequate shear reinforcement resulted in non-ductile behavior. Additionally gas-pressure weld splices of reinforcing bars failed.

Most pile failures are associated with liquefaction of soil which can result in buckling of the pile, loss of pile friction capacity, or development of pile cracking and hinging. Hinging may be at the cap location or at an interface between soil layers of differing lateral stiffness, Figure 3-4. Damage to piles in the soil may also occur about 1 – 3 pile diameters below grade in non-liquefying soils due to vibrational response of the wharf, Priestley et al (1998). Meyerson et al. (1992) present a state-of-the-art approach for evaluating pile buckling capacity for conditions of liquefaction of a soil layer and also determining allowable lateral deformation capacity. Most buckling occurs when the zone of liquefaction extends to the surface with the water table at the surface producing a large unsupported length. An axial transition load exists such that at less than that load the pile will not buckle.. Typically the transition load is at one-fourth to one-third of the ultimate bearing capacity. Flexible piles will tend to try to conform to soil movement and will have large curvatures at the interface between liquefied and non-liquefied material. Meyerson et al. present dimensionless curves relating pile lateral displacement capacity before formation of a hinge as a function of pile characteristic length. Yoshida and Hamada report on 2 case studies of piles beneath buildings which were damaged by liquefaction during the 1964 Niigata Earthquake. The first building, Building A, was a three-story reinforced concrete building and is shown in Figure 3-5. The piles in this case were end-bearing piles. The piles exhibited tensile cracks and concrete crushing. Pile number 2 exhibited a total disintegration of concrete probably from a lack of adequate confining steel. The second building designated as Building S is also a three-story reinforced concrete building and is shown in Figure 3-6. The piles in this case were friction piles. All piles in both buildings were damaged by the lateral deformation associated with the liquefaction. Generally pier piles will develop hinges first at the pile cap; however this may not be the case for wharves having piles with much shorter freestanding lengths.

Priestley et al. (1996) contains an extensive report on the causes of bridge damage much of which is relevant to piers. In discussing damage to existing older bridges they note " All deficiencies tend to be a natural consequence of the elastic design philosophy



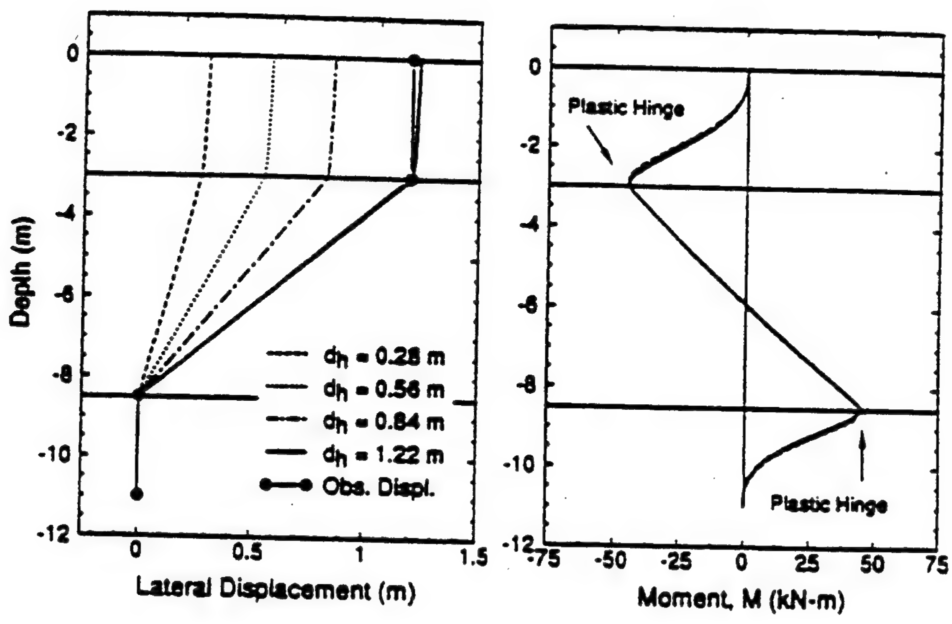
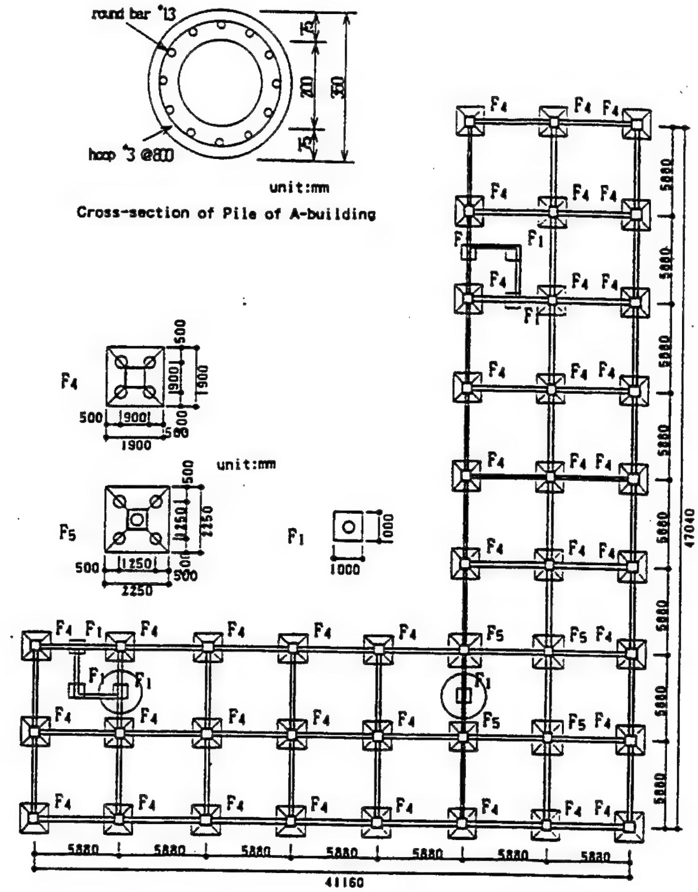
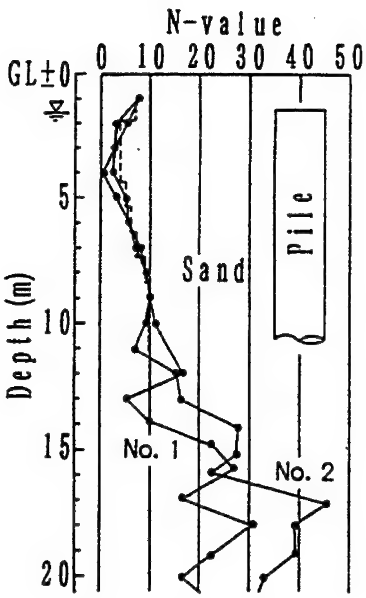


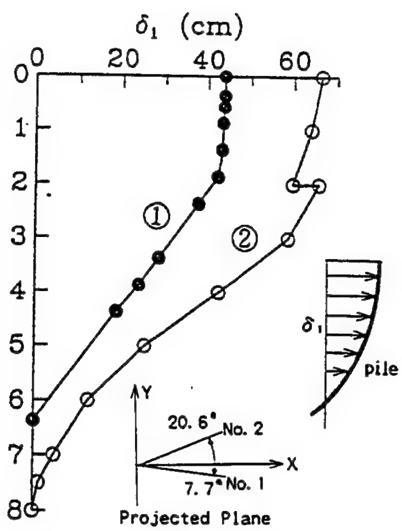
Figure 3-4. Typical pile damage in liquefaction zone.



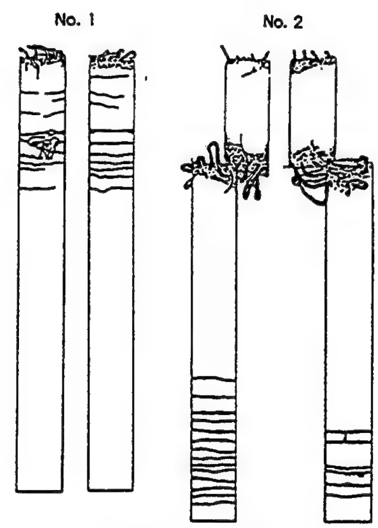
Cross-section of Pile of A-building



Soil Profiles and N-value Distribution



Deflection Pattern of Piles of A-building



Crack Pattern of Piles of A-building

Figure 3-5. Building A pile damage, Niigata Earthquake.

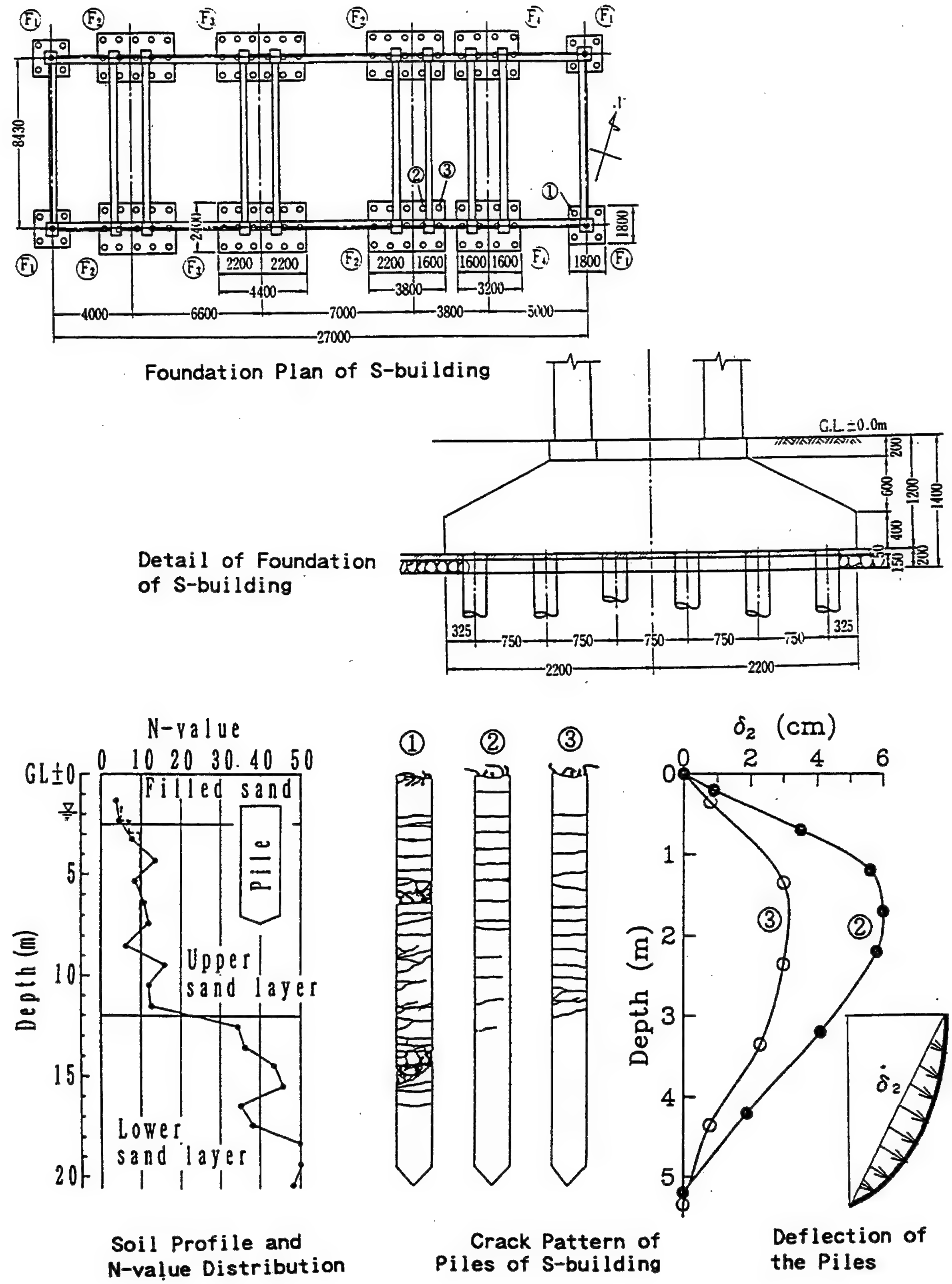


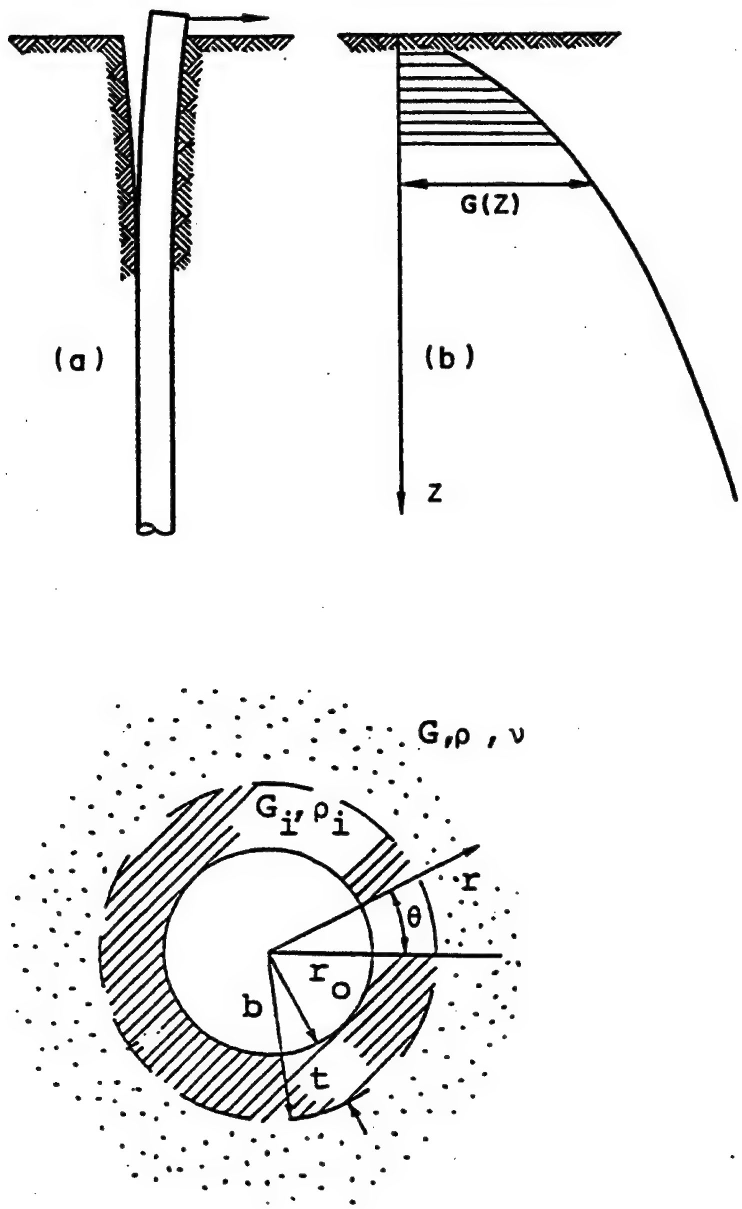
Figure 3-6. Building S pile damage, Niigata Earthquake.

almost uniformly adopted for seismic design of bridges prior to 1970, and still used in some countries notably Japan." Seismic deflections were underestimated in part by use of gross rather than cracked member stiffness. Seismic design forces were low and the ratio of seismic to static loads was incorrect resulting in erroneous moment patterns. Points of contraflexure were mislocated resulting in premature termination of reinforcement. Adjacent frames of bridges experienced out-of-phase-motion, with displacements often exceeding member supports. Soft soils amplified motion and liquefaction caused loss of pile support. Pounding of bridge members can impart high impact forces. Column longitudinal reinforcement was often lap-spliced immediately above the foundation with an inadequate development length of 20 bar diameters. " Displacement ductility factors as high as μ = 6 to 8 may be needed in some cases. At ductility levels of 2-3, concrete compression strains in the plastic hinge regions exceed the unconfined strain capacity and spalling of the cover concrete occurs. Unless the core concrete is well confined by closely spaced transverse hoops or spirals, the crushing rapidly extends into the core, the longitudinal reinforcement buckles, and rapid strength degradation occurs.." Cap beam failures were attributed to low shear capacity, early cutoff of negative top beam reinforcement, and insufficient anchorage of large diameter cap reinforcement in the column. Development lengths of 3 to 5 feet for #18 bars was shown to be inadequate in the Loma Prieta earthquake resulting in large flexural cap beam cracks. Rupture of #18 bars bent on a 18-inch radius. "Problems can be expected for columns with longitudinal reinforcement anchorage provided by 90 degree hooks bent away from the column axis, creating an unfavorable tension field in the joint region.. ... Current analysis indicates that considerable amount of vertical and horizontal shear reinforcement is required in the joint region, but are commonly omitted in older designs.

Priestley and Seible (1997) discuss aspects of analysis and design of piles for bridge structures, much of which is directly relevant to wharves and piers. Japanese research, Kubo, 1969 and Hayashi, 1974 reports that piles move with the soil during elastic portions of motion. Locations of potential failure occur at the point of sharpest curvature in the pile top, at just below the mud line, and at a depth in the soil profile at high curvature. Piles tend to move with the soil such that the region of maximum displacement slope variation in the soil field controls the pile response. The extent of this action depends on the soil stiffness and the pile stiffness. Batter piles exert large reactions on the pier structure which may have detrimental effect on the pile cap. The pier should be structurally separated from the abutment to provide isolation.

Characterization Of Soil Forces Acting On Piles

Novak (1991) gives a state-of-the-art paper on pile dynamics in which he discusses causes of damage to piles such as liquefaction and earth movement. He discusses theoretical studies which develop dynamic soil-pile interaction; he shows that there is a cylindrical boundary zone around a pile which undergoes nonlinear behavior. Pile soil separation is possible under lateral load, Figure 3-7. The length of the pile separation, L_s, is a function of the lateral deformation:



Cylindrical boundary zone around pile

Figure 3-7. Pile separation and zone of plastic behavior.

$$L_s / d = 260 \ \Delta/d \qquad (3\text{-}1)$$

where

Δ the amount of lateral deformation , $0.001 \le \Delta/d \le 0.005$
d Pile diameter

Wolf and Weber (1986) show the difference in horizontal stiffness and damping for alternative modeling assumptions. Figure 3-8a shows a linear model with soil tension. Figure 3-8b allows soil separation which is seen to reduce damping. Figure 3-8c allows soil separation and slipping of the pile in the soil which reduces both horizontal stiffness and damping. Large displacements require nonlinear representation of the soil around the pile. To account for gapping, slippage and friction lumped mass finite element models evolved as the most often used approach. Soil resistance deflection relationships known as p-y curves were developed. Figure 3-9 is a typical p-y curve. To account for pile separation the soil reaction displacement curve shown in Figure 3-10 has been used. Figure 3-11 shows cyclic loading p-y curves for sand and clay.

Yoshida and Hamada (1991) report on the Japanese Highway Bridge Code which establishes the subgrade modulus reaction k for use with piles:

$$k = 0.2 * 28N * D^{-0.35} \ (\mathrm{kgf/cm^3}) \qquad (3\text{-}2)$$

where

D the diameter of the pile in cm
N Japanese penetration test N value of blowcounts

The spring constant is found by multiplying the diameter of the pile times k times the pile length between springs.

Martin and Lam (1995) present a recent state-of-the-art summary of the design of pile foundations. They show that a nonlinear soil model is required to capture the lateral behavior of a pile. A Winkler Spring Beam-Column representation with nonlinear springs is shown to be an acceptable method for computing pile behavior to lateral loads. They have reviewed procedures for computing the required soil load-deformation relationship to characterize the spring properties and found the American Petroleum Institute (1994) procedure to be the accepted common practice. This procedure is found in a recent recommended practice and is approved by the American National Standards Institute.

The origin of the API equation for sand evolved from work by Reese, Cox, and Koop (1974) who established a set of equations based on the forces associated deformation of a soil wedge and the lateral deformation of a rigid cylinder into soil. They established the early shape of the soil load deflection p-y curve based on the soil subgrade

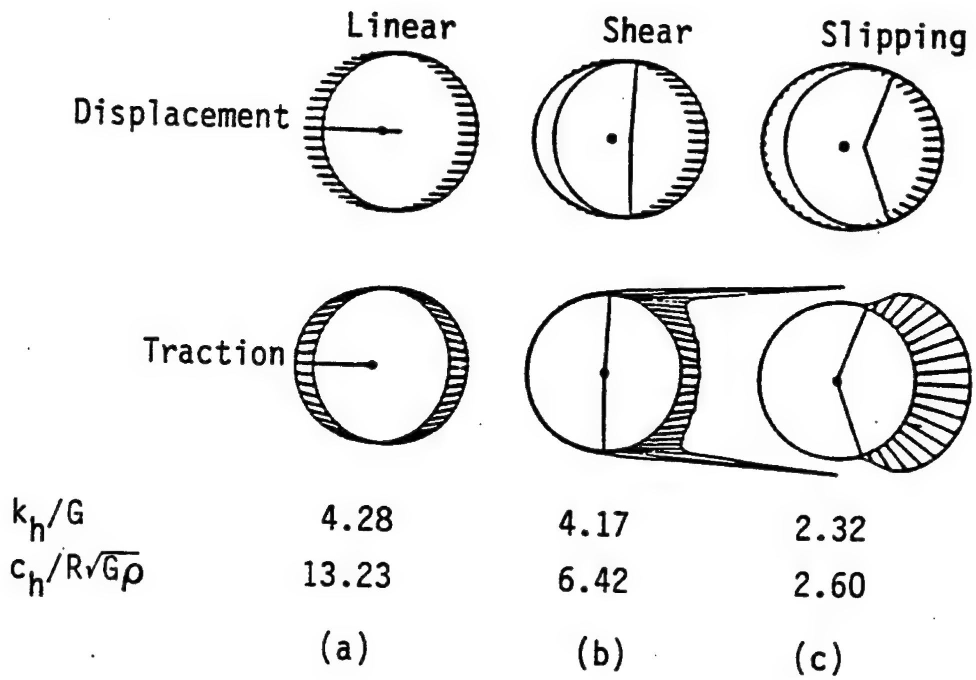


Figure 3-8. Influence of model on behavior.

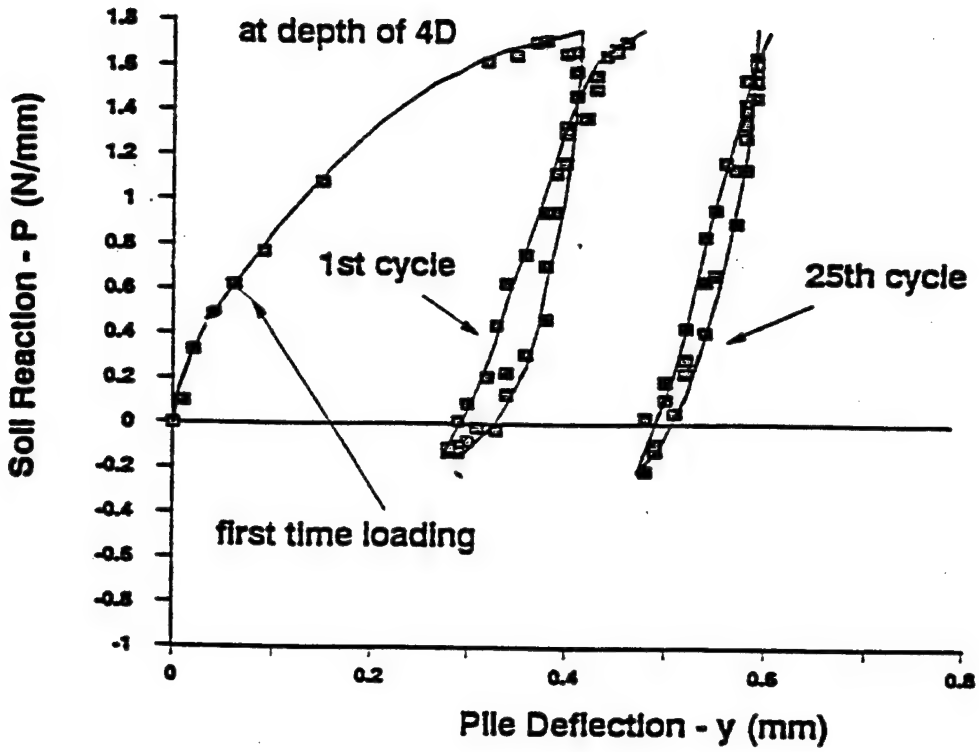


Figure 3-9. Typical p-y curve.

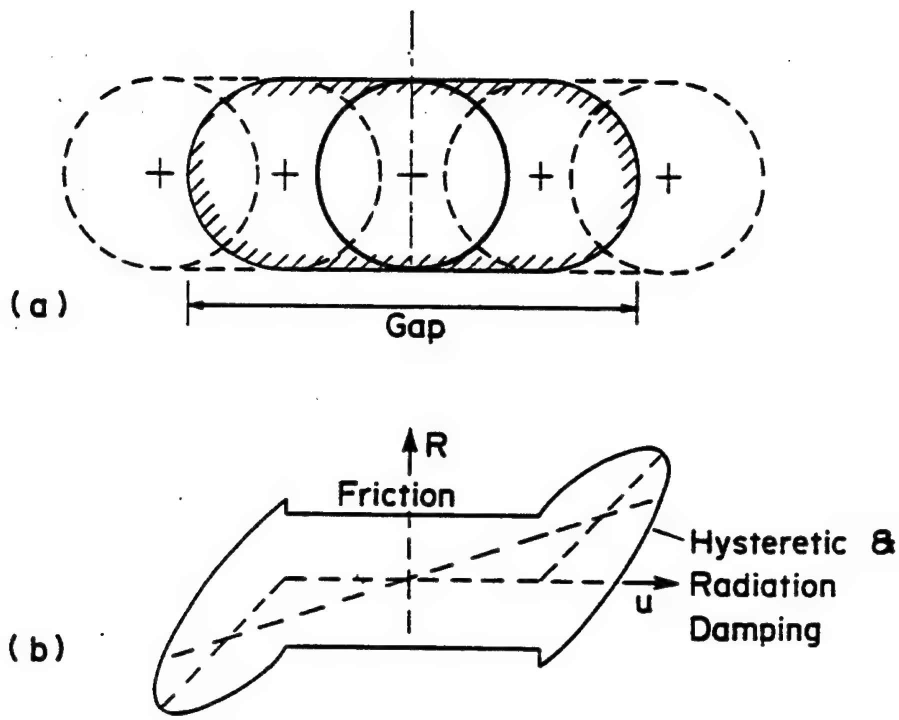


Figure 3-10. Gapping and hysteretic behavior.

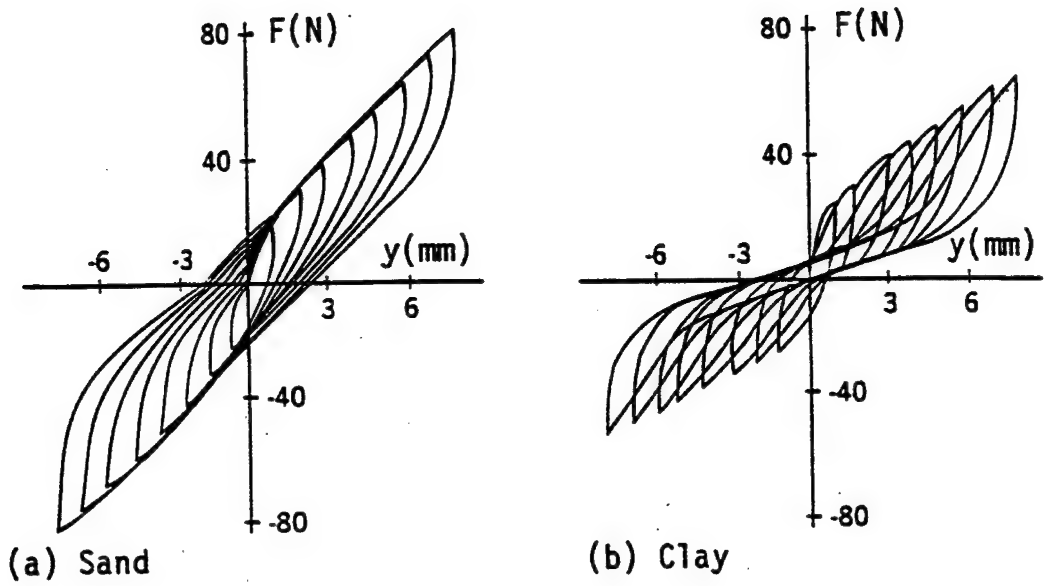


Figure 3-11. Cyclic behavior for sand and clay.

modulus. The procedure was modified by Bogard and Matlock (1980) principally as a simplification by consolidation of terms. The shape of the p-y curve was finally based on work by Parker and Reese (1970). O'Niell and Murtcheson (1983) wrote an excellent summary of the development of the procedure for constructing p-y curves and performed a comparison study showing the API equation as having least cumulative error in comparison with experimental data from full scale pile tests, although one must consider that this is still a very approximate procedure.

The American Petroleum Institute (1994) recommended practice for offshore platforms gives guidance in determining p-y curves. That information is reported verbatim in the following sections:

> **Lateral Bearing Capacity for Soft Clay.** For static lateral loads the ultimate unit lateral bearing capacity of soft clay p_u has been found to vary between 8c and 12c except at shallow depths where failure occurs in a different mode due to minimum overburden pressure. Cyclic loads cause deterioration of lateral bearing capacity below that for static loads. In the absence of more definitive criteria, the following is recommended. The value of p_u increases from 3c to 9c as X increases from 0 to X_R according to:
>
> $$p_u = 3c + \gamma' X + J c X / D \qquad (3\text{-}3)$$
>
> and
>
> $$p_u = 9c \text{ for } X \geq X_R \qquad (3\text{-}4)$$
>
> where:
>
> p_u ultimate resistance, in stress units
> c undrained shear strength of undisturbed clay soil samples, in stress units
> D pile diameter
> γ' buoyant unit weight of soil, in weight density units
> J dimensionless empirical constant with values ranging from 0.25 to 0.5 having been determined by field testing. A value of 0.5 is appropriate for Gulf of Mexico clays.
> X depth below soil surface
> X_R depth below soil surface to bottom of reduced resistance zone. For a condition of constant strength with depth, Equations 3-3 and 3-4 are solved simultaneously to give:
>
> $$X_R = 6D / ((\gamma' D / c) + J) \qquad (3\text{-}5)$$
>
> Where the strength varies with depth, Equations 3-3 and 3-4 may be solved by plotting the two equations, i.e., p_u vs. depth. The point of first intersection of two equations is taken to be X_R. These empirical relationships may not apply where strength variations are erratic. In general, minimum values of X_R should be about 2.5 pile diameters.

Lateral soil resistance-deflection relationships for piles in soft clay are generally nonlinear, Figure 3-12. The p-y curves for the short-term static load case may be generated from the following table:

p/p_u	y/y_c
0.00	0.0
0.50	1.0
0.72	3.0
1.00	8.0
1.00	∞

where:

- p — actual lateral resistance, in stress units
- y — actual lateral deflection
- y_c — $2.5\ \varepsilon_c D$
- ε_c — strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples

For the case where equilibrium has been reached under cyclic loading, the p-y curves may be generated from the following table:

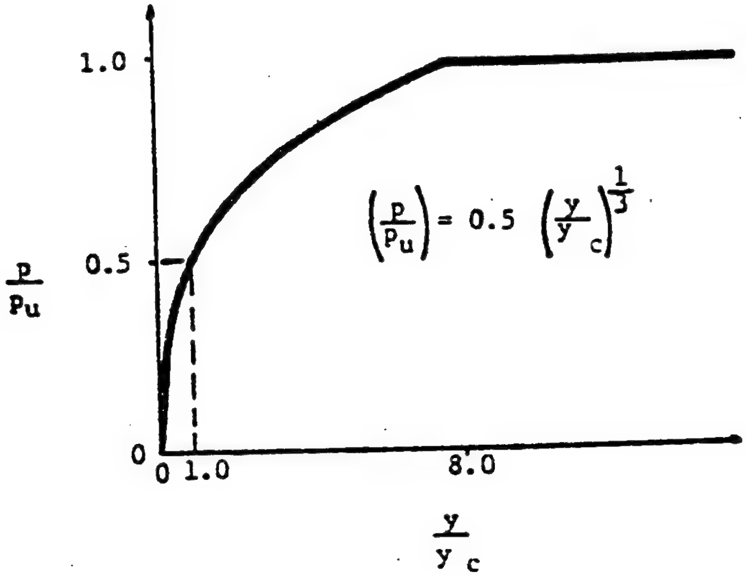
$X>X_R$		$X<X_R$	
p/p_u	y/y_c	p/p_u	y/y_c
0.5	1.0	0.5	1.0
0.72	3.0	0.72	3.0
0.72	∞	$0.72X/X_R$	15.0
		$0.72X/X_R$	∞

Lateral Bearing Capacity for Stiff Clay. For static lateral loads, the ultimate bearing capacity, p_u, of stiff clay (c > 96 kPa or 1 Tsf) as for soft clay would vary between 8c and 12c. Due to rapid deterioration under cyclic loadings, the ultimate static resistance should be reduced for cyclic design considerations. While stiff clays also have nonlinear stress-strain relationships, they are generally more brittle than soft clays. In developing stress-strain curves and subsequent p-y curves for cyclic loads, consideration should be given to the possible rapid deterioration of load capacity at large deflections for stiff clays.

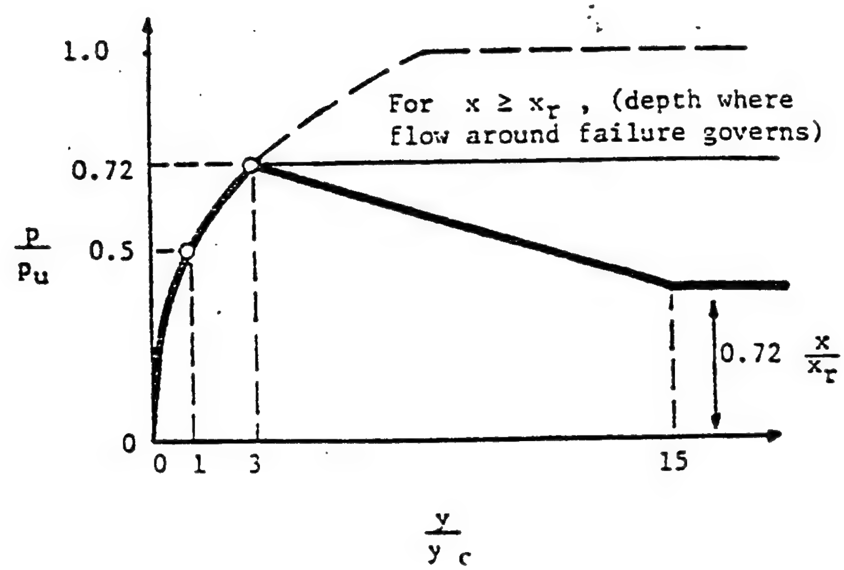
Lateral Bearing Capacity for Sand. The ultimate lateral bearing capacity for sand has been found to vary from a value at shallow depths determined by Equation 3-6 to a value at deep depths determined by Equation 3-7. At a given depth the equation giving the smallest value of Pu should be used as the ultimate bearing capacity.

$$P_{us} = (C_1X + C_2D)\,\gamma' X \qquad (3\text{-}6)$$

$$P_{ud} = C_3 D\ \gamma'\ X \qquad (3\text{-}7)$$



a. Static loading



b. Cyclic loading

Figure 3-12. Shape of p-y curves for soft clay

where

Pu ultimate resistance (force/unit length) (s=shallow, d=deep)
γ' buoyant soil weight, in weight density units
X depth
ϕ' angle of internal friction in sand
Cl Coefficient determined from Figure 3-13 as a function of ϕ'
C2 Coefficient determined from Figure 3-13 as a function of ϕ'
C3 Coefficient determined from Figure 3-13 as a function of ϕ'
D average pile diameter from surface to depth

The lateral soil resistance-deflection (p-y) relationship for sand is also nonlinear and in the absence of more definitive information may be approximated at any specific depth X, by the following expression.

$$P = A\, p_u \tanh [\, (k\, X\, y\,)/(A\, p_u)] \qquad (3\text{-}8)$$

where

A factor to account for cyclic or static loading continued.

$A = 0.9$ for cyclic loading.

$A = (3.0 - 0.8X/D) \geq 0.9$ for static loading.

p_u ultimate bearing capacity at depth X in units of force per unit length
k initial modulus of subgrade reaction in force per volume units. Determine from Figure 3-14 function of angle of internal friction.
y lateral deflection
X depth

Pile group. Consideration should be given to the effects of closely spaced adjacent piles on the load and deflection characteristics of the pile group. Generally, for pile spacing less than eight diameters, group effects may have to be evaluated.

For piles embedded in clays. the group capacity may be less than a single isolated pile capacity multiplied by the number of piles in the group; conversely, for piles embedded in sands, the group capacity may be higher than the sum of the capacities of the isolated piles. The group settlement in either clay or sand would normally be larger than that of a single pile subjected to the average pile load of the pile group.

For piles with the same pile head fixity conditions and embedded in either cohesive or cohesionless soils, the pile group would normally experience greater lateral deflection than that of a single pile under the average pile load of the corresponding group. The major factors influencing the group deflections and load distribution among the piles are the pile spacing, the ratio of pile penetration to the diameter, the pile flexibility relative to the soil, the dimensions of the group, and the variations in the shear strength and stiffness modulus of the soil with depth.

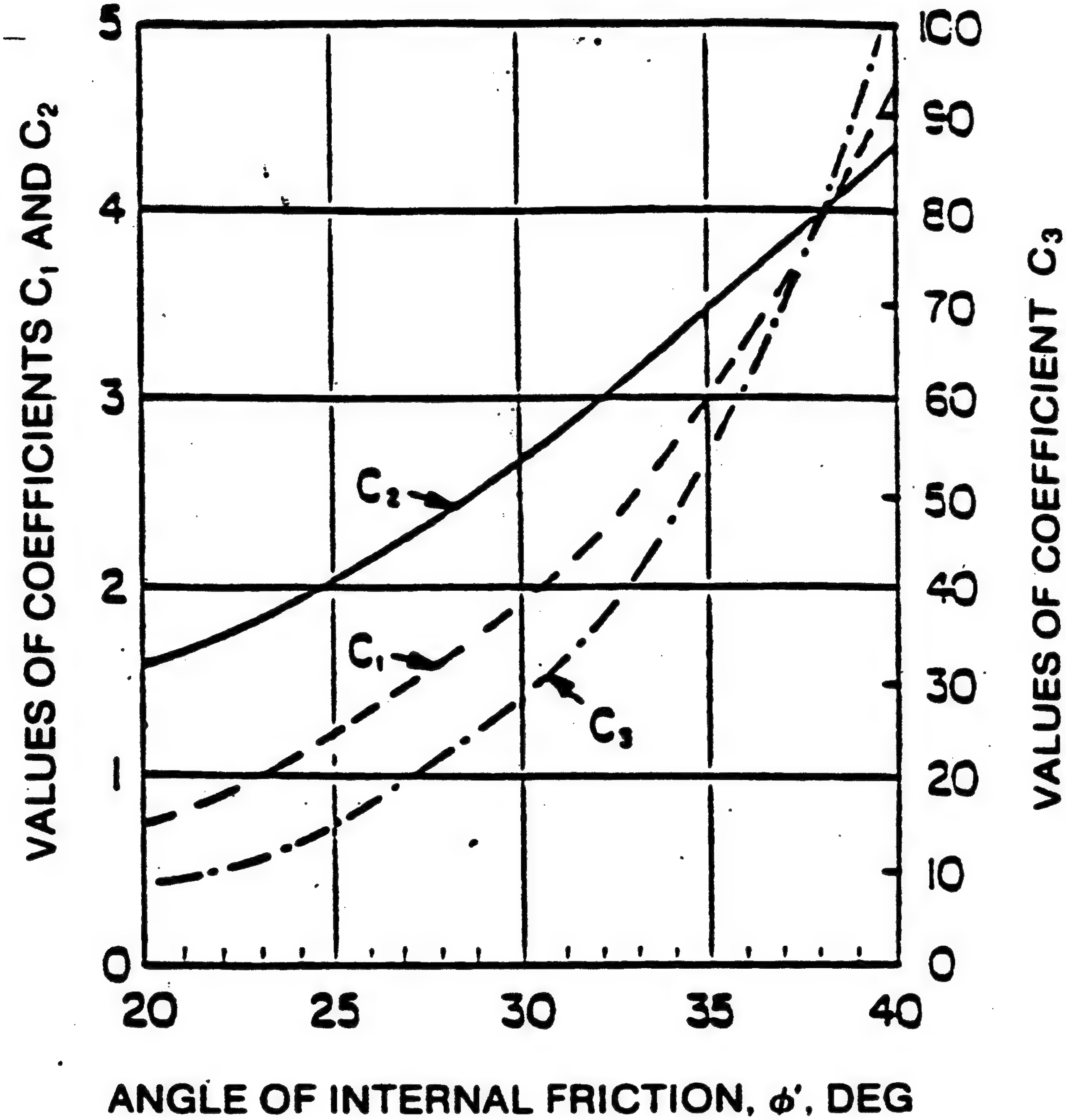


Figure 3-13. API Coefficients for sand.

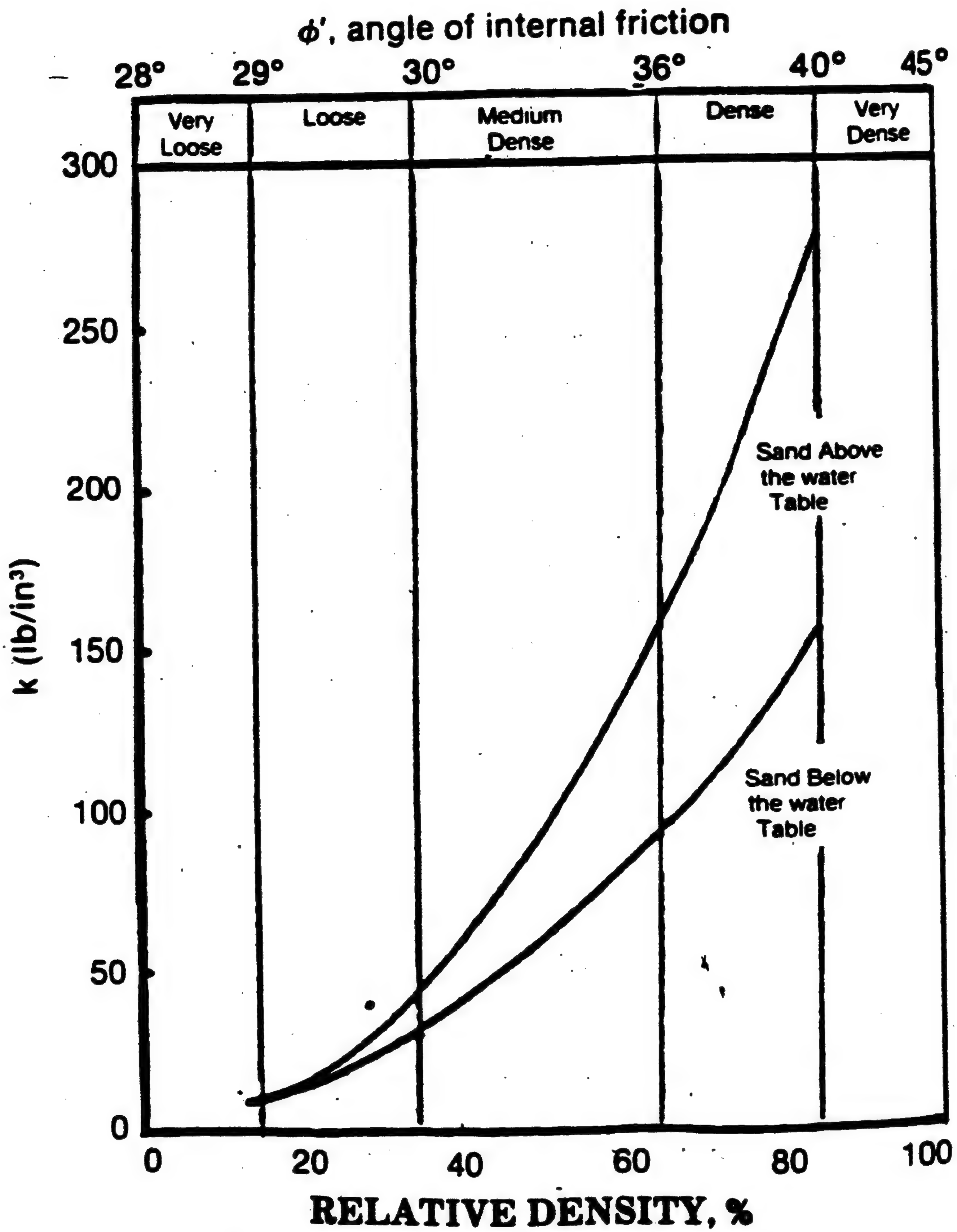


Figure 3-14. API initial modulus of subgrade reaction.

It has been noted that piles spaced 5 pile diameters apart do not exhibit a significant group effect.

Soil Properties

The soil properties which influence the pile lateral deflection and are required for definition of a spring model are as follows:

Cohesionless Soils

γ' buoyant soil weight, in weight density units

ϕ' angle of internal friction in sand

Cohesive Soils

ε_c strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples

γ' buoyant unit weight of soil, in weight density units

c undrained shear strength of undisturbed clay soil samples, in stress units

J dimensionless empirical constant with values ranging from 0.25 to 0.5 having been determined by field testing. A value of 0.5 is appropriate for Gulf of Mexico clays.

The following soil properties are taken from the NCEL Handbook for Marine Geotechnical Engineering (1985).

Properties for Cohesionless Soil

Type	Standard Penetration Blow Count, N	ϕ (Degrees)	Relative Density, D_r (%)	Effective Unit Weight, γ_b (lb/cu ft)
Very Loose to loose	<10	28-30	0-35	45-55
Medium Dense	10-30	30-36	35-65	55-65
Dense	30-50	35-42	65-85	60-70
Very Dense	50 +	40-45	85-100	60-70

Properties of Cohesive Soils

Type	Undrained Shear Strength, (Lb/sq in)	Strain at 50% maximum stress ε_c	Effective Unit Weight, γ_b (lb/cu ft)
Unconsolidated clays	0.35-1.0	2	20-25
Normally consolidated soils at depth z , inches.	1.0 + 0.0033z	2-1	25-50
Overconsolidated soils based on consistency:			
medium stiff	3.5-7	1.0	50-65
stiff	7-14	0.7	50-65
very stiff	14-28	0.5	50-65
hard	over 28	0.4	50-65

Values of ε_c can be estimated from the following table when other data is not available:

Shear Strength lb/sq ft	ε_c %
250-500	2.0
500-1000	1.0
1000-2000	0.7
2000-4000	0.5
4000-8000	0.4

Pier Analysis

A typical pier was studied, Ferritto (1997). The pier was 80 feet across and a typical bent is illustrated in Figure 3-15. The 9 piles are 24-inch octagonal prestressed piles discussed in the previous section. The pier was modeled by a 2-dimensional analysis. A static lateral force push over analysis was applied in conjunction with the standard vertical dead and live loads. It was found that the pier resisted 261 kips applied horizontally before collapse began. Yielding of the piles at the deck level with the formation of the first hinge; a second hinge developed at the mud line depth which produced a collapse mechanism. The structure underwent large gradual deformation near failure. The structure was analyzed using the El Centro earthquake record as a dynamic lateral load function. The structure was able to undergo large deflection without the

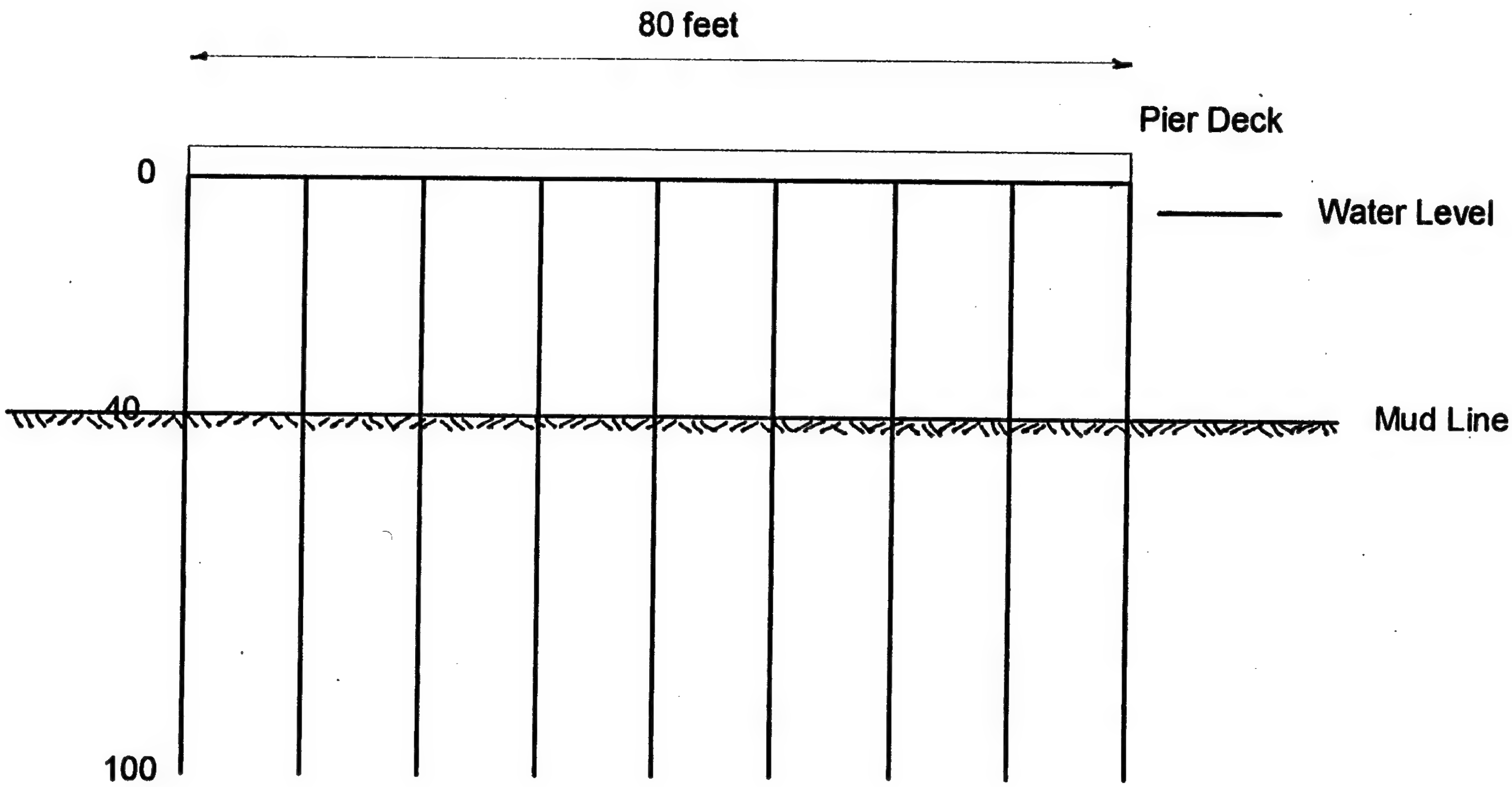


Figure 3-15. Typical pier used in 2-dimensional analysis.

occurrence of a computational instability indicating collapse. As a practical limit, a 10-inch displacement was set as an effective limit. This occurred at a lateral load of about 0.75g.

The pier was modified by the inclusion of two 24-inch octagonal prestressed concrete batter piles as shown in Figure 3-16. The extent of restraint provided by the batter piles is a function of soil restraining the piles. Modeling of batter piles involves not only the horizontal py soil resistance but also the tz vertical soil resistance and the amount of q-w end-bearing. These will be discussed in the next section. For this case, the batter piles had a significant stiffening effect on the structure. A static lateral force push over analysis was performed and a lateral force of 648 kips was found to cause. As loading increased the batter pile in tension reached its failure capacity first. When this pile failed load was transferred to the remaining batter pile which failed causing failure of all the remaining piles. The structure performed in a brittle manner such that collapse occurred immediately after the first pile failed. This structure was also analyzed using the El Centro earthquake record as a dynamic lateral load function. Collapse occurred at a peak horizontal acceleration of about 0.9g. The structure had substantially reduced lateral displacements compared to the pier without batter piles. Again the onset of batter pile failure resulted in the rapid collapse of the structure and was initiated by exceeding pile tensile limits.

Batter Piles

Previous sections focused on the lateral resistance of vertical; to fully understand the behavior of a batter pile it is necessary to review the axial pile soil skin friction (tz component) and the pile end bearing (qw component). For a vertical pile the axial and end bearing components are also vertical. Figure 3-17 shows the force components acting on a vertical pile. The spring elements are intended as visual aids to represent the forces acting along the entire length of the pile. Ultimate capacity of a vertical pile is given by:

$$Q = f A_s + qA_p \tag{3-9}$$

where

f	Unit skin friction, tz
A_s	Area side surface of pile
q	Unit end bearing capacity, qw
A_p	Area of end of pile

For cohesive soils the skin friction is

$$f = a\,c \tag{3-10}$$

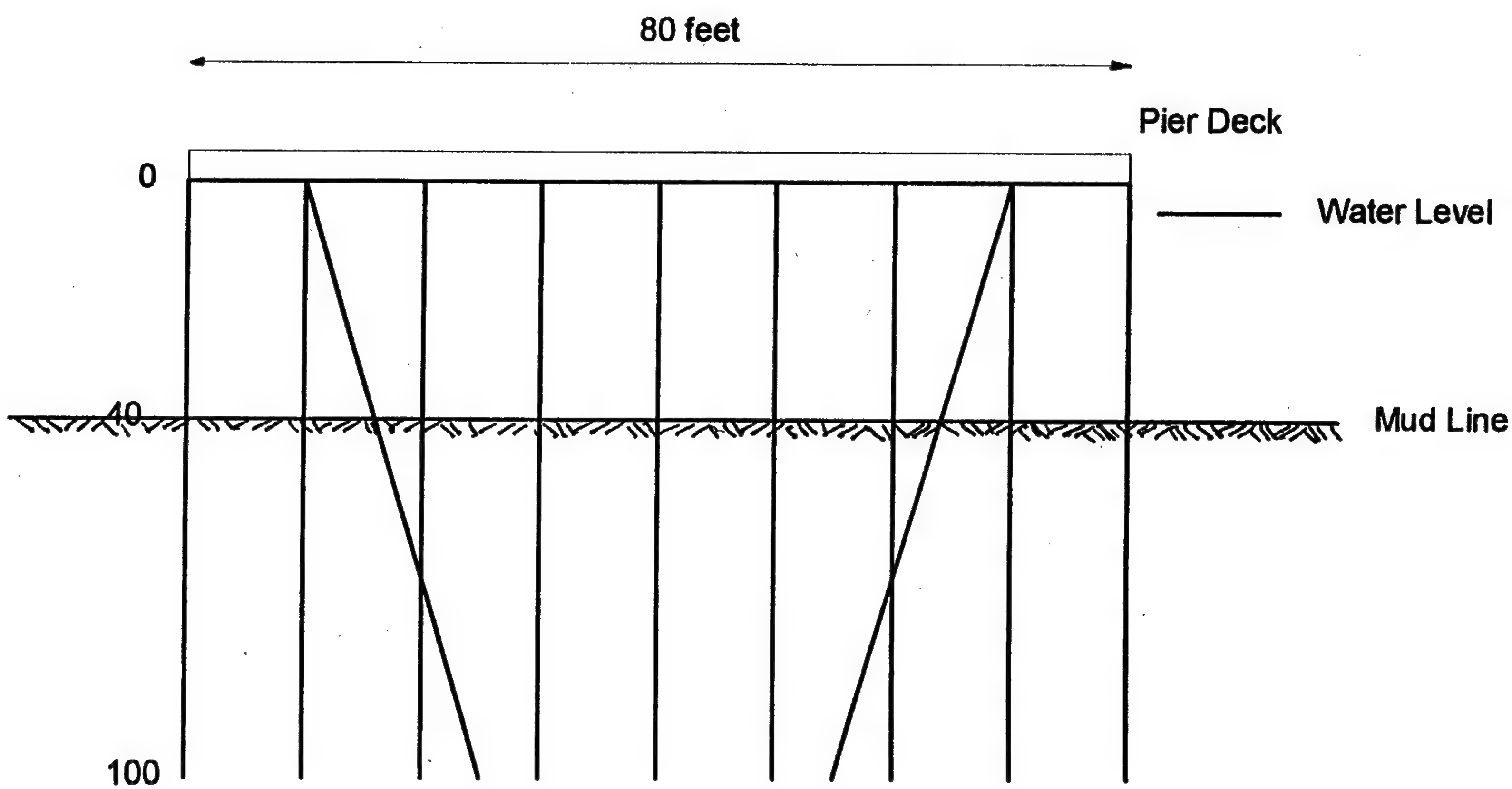


Figure 3-16. Typical pier with batter piles used in 2-dimensional analysis.

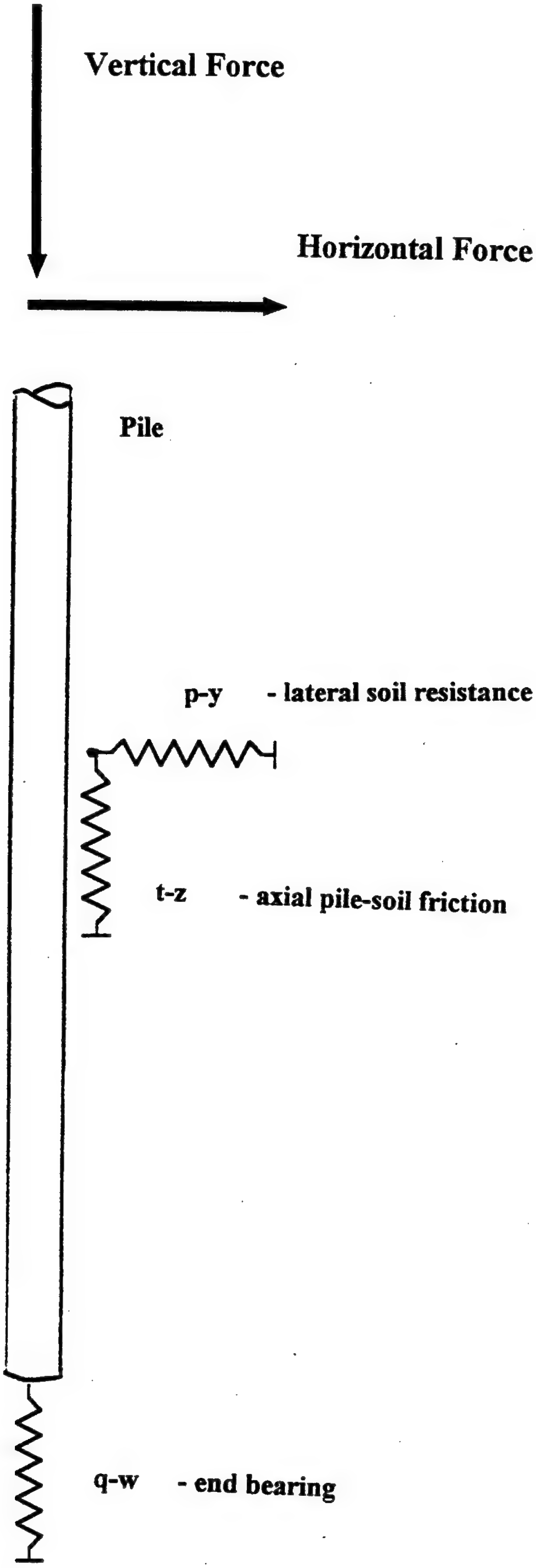


Figure 3-17. Vertical pile in soil showing restraints.

where

$$a = 0.5\,\psi^{-0.5} \quad \psi \le 1.0$$
$$a = 0.5\,\psi^{-0.25} \quad \psi > 1.0 \qquad (3\text{-}11)$$

and

c	Undrained shear strength
ψ	c/p_o'
p_o'	Effective overburden at point in question

For cohesive soils the end bearing unit stress of piles is:

$$q = 9\,c \qquad (3\text{-}12)$$

For cohesionless soils the unit skin friction is:

$$f = K\,p_o' \tan\delta \qquad (3\text{-}13)$$

where

K	dimensionless coefficient of lateral earth pressure, usually = 1
δ	friction angle between the soil and pile

For cohesionless soils the unit end bearing stress is:

$$q = p_o' N_q \qquad (3\text{-}14)$$

API (1994) gives procedures for computing tz axial force-deflection curves and qw end bearing-deflection curves which define the soil resistance pictured in Figure 3-17. For comparison consider the following for cohesionless soils:

Lateral	Vertical
$pu/D = (c1\,X + C2\,D)\,\gamma'\,X/D$	$f = K\,p_o'\tan\delta\,D$

for a depth of 10 feet, 18-inch pile diameter

$pu/D \cong (14)\,\gamma'\,X$	$f \cong (.7)\,\gamma'\,X$

For cohesive soils

Lateral	Vertical
pu ≅ 10 c	q ≅ 1 c

From the above it may be seen that the py lateral resistance is much greater than the tz axial skin friction. For this reason it is logical that most of the lateral resistance of a pile is mobilized in the near surface region of a pile to a depth of 5 to 10 pile diameters. To resist vertical loads without end bearing requires long pile development lengths. End bearing is a significant component in pile capacity. The magnitude of the end bearing resistance is on the same order as the lateral resistance. The capacity of a pile in tension is less than in compression and less than in lateral resistance. The above are general observations and the ratio of lateral capacity to tension or compression capacity depends on a number of factors:

- Length of the pile
- Flexural and shear strength of the pile
- Vertical distribution of soil strength
- Acceptable lateral displacement limits.

Having reviewed the fundamentals of vertical pile behavior, it is now possible to discuss batter piles.

Figure 3-18 illustrates a batter pile based rotation of a vertical pile. For simplicity the forces are kept normal and axial to the pile. Consider as the tz axial resistance of the soil goes to zero the pile would tend to slip out of the ground with minimal axial loading in tension. The modeling of a pile by finite element representation must accurately capture this interaction. In a finite element model, it is possible to use spring/truss elements to model the soil resistance. Use of horizontal and vertical springs to model the components of py and tz resistance would introduce a major problem of how to combine these elements. The axial tz acts independent of the normal py and must allow pile slippage. End bearing is an axial component. After a number of trial iterations of various models, it was found that the most accurate and easiest to use is one with spring elements axial and normal to the batter pile. Additionally the end bearing must be an axial spring. All py, tz and qw load deflection curves are based on the depth of the element below the ground surface.

An analysis was performed on a batter pile with a 1 horizontal to 2 vertical slope driven to a depth of 50 feet and loaded laterally at a height of 3 feet above the ground. The pile was a 24-inch circular pipe pile in medium sand. The soil py and tz curves were calculated at intervals of 1 foot using the equations in API (1994). Normal and axial soil springs were spaced at 1-foot intervals for the first 20 feet and then at 2-foot intervals. The soil springs utilized bilinear material properties. A vertical load of 20 kips was used. The lateral capacity of the batter pile when pulled horizontally in a direction away from

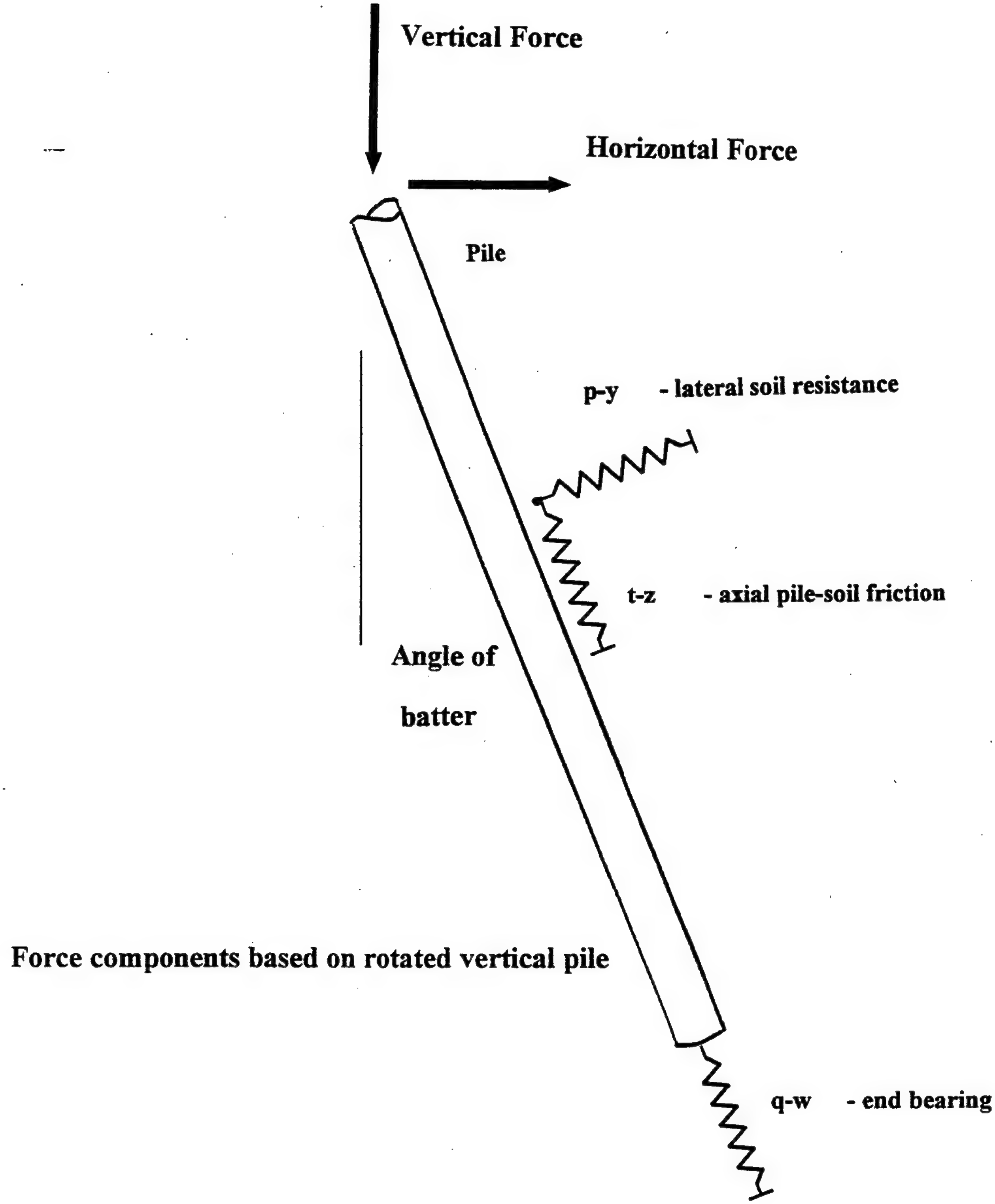


Figure 3-18. Batter pile in soil showing restraints.

the batter was 12 kips compared to 36 kips when pulled horizontally in the direction of the batter. The lateral capacity of an equivalent vertical pile was 20 kips.

From a series of studies of a batter pile it is concluded that the lateral load capacity of the batter pile is dependent upon the vertical load of the pile. When the lateral load acts horizontally opposite to the direction of the batter, a component of this load acts on the pile axially in tension. The vertical load offsets this effect and can increase pile capacity up to a limit. This is generally the direction of lower pile capacity. When the lateral load acts horizontally toward the direction of the batter the vertical load resists the moment caused by the lateral load and reduces deflection.

The vertical distribution of pile soil reaction is a function of pile length. The longer the pile the greater the friction along the pile and the less in end bearing. Since stiffness increases with depth more force is transferred into the soil at deeper depths (assuming pile compression is not significant).

It should be noted that connections to adjacent piles in group should be capable of developing the calculated tension including earthquake forces, but not less than a tension equal to one-half the compression load less the dead weight in the pile. Unless special provisions are made for the difficulties of installation and the effects of diminution of the hammer blow on the capacity, keep the slope of the batter piles to one horizontal to two vertical or steeper (preferably 1 horizontal to 2.5 vertical).

P - Delta Effect

One element of structural analysis should be noted- that of the P - delta effect associated with columns in frames. This is usually thought of as a secondary effect composed of the additional moment which is imposed on a column by the axial load acting on a moment arm caused by the lateral deflection of the top of the column. For piers and bridge structures this can be significant. Duan and Cooper, 1995 discuss this extensively and conclude that the effect should be included in seismic analysis. Note that the peak lateral resisting force is reduced.

Use of Elastic Response Spectra Techniques

The above sections have illustrated the need for nonlinear representation of the soil-pile interface and the p-Δ effects. Use of linear response spectra and force reduction factors in building codes has been common practice. The use of linear techniques should be applied rationally. In pier design, such linear techniques should be used when appropriate and avoided where there is significant nonlinear yielding such as where soft soils are present. The engineer has a variety of tools which are tailored to the complexity of the problem.

Design Performance And Earthquake Levels

Performance Goal

The goal of this criteria is to standardize the seismic design of piers and wharves providing an acceptable uniform level of seismic safety for all waterfront locations. This criteria is intended to produce a level of design in piers and wharves such that there is a high probability the structures will perform at satisfactory levels throughout their design life. Waterfront structures have been classified as essential structures. Although the criteria focus on the regions where seismicity is highest such as on the West Coast, it is applicable to other areas as well. Structures located in areas of high seismicity shall be designed:

- To resist earthquakes of moderate size which can be expected to occur one or more times during the life of the structure without structural damage of significance. This represents a condition of expected repeated loading.

- To resist major earthquakes which are considered as infrequent events maintaining environmental protection and life safety, precluding total collapse but allowing a measure of controlled inelastic behavior which will require repair. . This represents a condition of expected loading to occur at least once during the design life of the structure. In reality most structures end up being used well beyond their design life.

- To preclude major spills of hazardous and polluting materials for very rare earthquakes The intention is to prevent spills on fuel piers. This may be accomplished by installation of cutoff valves, and containment to limit the size of the spill and prevent its spread. It may also be accomplished by increased strengthening of components.

- To utilize economic/risk analysis where necessary. This is intended to allow the designer freedom to consider economic analysis of alternative load conditions in the infrequent case where a local fault dominates a site and is capable of very high ground motions. Such a condition requires specialized extensive evaluation of the site hazard.

- To consider liquefaction as a major waterfront problem. The designer shall consider liquefaction factors of safety in design of remedial measures of backfill. Rigid adherence to developing fixed factors of safety may not be economically achievable. The intent is to place more credence in the expected deformations and consequences of liquefaction which will occur rather than the simple factor of safety. Assurance of limited deformations shall be given precedence over a factor of safety. The designer shall have the option of using current technology to demonstrate that settlements and lateral deformations are sufficiently limited to insure structural performance and factors of safety lower than limit values may be used. The term *current technology*

refers to the use of procedures for the computation of vertical and lateral deformations.

In general all waterfront construction which supports fuel transfer operations (excluding piers and wharves) falls within the category of essential construction. The policy is to minimize downtime for these facilities. Determination of essential construction shall be determined by the user in conjunction with the California State Lands Commission, Marine Facilities Division. Piers and wharves shall be considered essential construction. Emphasis shall be placed on minimizing downtime and interruption to essential functions.

Limit States

Level 1 Earthquake; Serviceability Limit State Two levels of design earthquake are considered. Level 1, has a high probability of occurrence during the life of the pier or wharf. Under this level of excitation the structure should satisfy the serviceability criterion of continued functionality immediately after the Level 1 earthquake. Any repairs required should be essentially cosmetic. Structural damage requiring repair is not permitted. Note that this does not imply a requirement for elastic response, which would limit concrete strains to about 0.001, and steel strains to yield strain. Concrete structures may be considered serviceable, without any significant decrement to structural integrity, provided concrete strains reached during maximum seismic response to the Level 1 earthquake do not cause incipient spalling of concrete cover, and if residual crack widths after the earthquake has ceased are sufficiently small to negate the need for special corrosion protection measures, such as crack grouting. Note that this latter requirement implies that significantly larger crack widths might momentarily exist during seismic response, since these have no effect on corrosion potential.

Thus the performance of potential plastic hinges should be checked under Level 1 earthquake to ensure that maximum material strains do not exceed the limits defined for Structural Performance Limit States.

Note that this check will normally only be required at the pile/deck interface, since curvatures will be higher there than in any potential in-ground hinges. Also, confinement of the cover concrete of potential in-ground plastic hinges by the lateral pressure developed in the soil increases the concrete strain at which spalling initiates, Budek et al (1997), increasing the corresponding serviceability curvatures, particularly for prestressed piles, where steel strains are unlikely to be critical.

Level 2 Earthquake; Damage-control Limit State Level 2 earthquake represents an earthquake with a low probability of occurring during the design life of the structure. Repairable damage to structure and/or foundation, and limited permanent deformation are acceptable under this level of earthquake

Since the design philosophy (discussed subsequently) is to restrict inelastic action to careful defined and detailed plastic hinges in piles, strain limits may again be used to define acceptable response. Distinction needs to be made between the pile/deck hinge and the in-ground hinge locations, since access to the latter will frequently be impractical after an earthquake. **As a consequence the in-ground hinge should satisfy serviceability criteria even under Level 2 earthquake. Thus the steel strain should not exceed 0.01.** However, as noted above, passive confinement by the soil increases the spalling strain, and a concrete strain of 0.008 may be adopted. The pile/deck connection hinge will have allowable concrete strains dependent on the amount of transverse reinforcement provided. As a consequence of these actions, the maximum strains calculated under a Level 2 earthquake must not exceed the strain capacities defined below under *Structural Criteria For Piers and Wharves.*

Level 4 Earthquake; Prevent Major Spill Wharves and piers on which hazardous materials are stored or used shall be capable of resisting a Level 4 earthquake without release of a major spill of hazardous materials.

Load Combinations

Two load combinations are to be considered.

$$(1 + k)(D + rL) + E \qquad (3\text{-}15)$$

$$(1 - k) D + E \qquad (3\text{-}16)$$

where D = Dead Load
L = Design Live Load
r = Live Load reduction factor(depends on expected L present in actual case typically 0.2 but could be higher)
E= Level 1 or Level 2 earthquake, as appropriate.
k= 0.5 * (PGA), where PGA is the effective peak horizontal ground acceleration.

The first equation, Equation 3-15, includes maximum gravity load effects and incorporates a realistic assessment of probable live load, while the second, Equation 3-16, includes minimum gravity loads. Influences of vertical acceleration are considered through the factor **k** which increases gravity load effects in Equation 3-15, but reduces the effects of gravity in Equation 3-16. The value of **k** is taken as 0.5 times the effective peak horizontal ground acceleration. This is based on the observation that peak vertical accelerations are generally less than peak horizontal accelerations. A typical value of

67% is often assumed. Further, peak vertical response is likely to be of very short duration and unlikely to coincide with peak horizontal response. The simplified approach suggested has been proposed for bridge design, Priestley et al (1996), and will be adequately conservative in most cases. An exception may occur when a large proportion of the seismic risk at the site comes from a fault closer than 10km from the site. In this case special studies should be undertaken to determine the peak vertical acceleration, and a value of 75% of effective peak vertical acceleration adopted for **k**. Alternatively inelastic analysis incorporating simultaneous excitation by vertical and horizontal acceleration records may be carried out.

Note that the main influence of the combination of gravity and seismic effects will be on the design of deck members, where the combined forces (moment, shear, etc.)from gravity and seismic effects must remain below the dependable strength, in accordance with capacity design principles. For the primary seismic resisting elements, which are typically the wharf or pier piles, gravity effects have comparatively little influence. Even at the serviceability limit state, the deformations corresponding to gravity loads will be small compared with those due to the Level 1 earthquake. Also, the variation in gravity axial load on piles implied by the difference between Equations 3-15 and 3-16 will not generally result in significant differences in lateral displacement capacity. As discussed in the section on pier and wharf response, seismic performance is judged primarily in terms of displacement capacity and demand, with force levels being of only secondary significance.

Note further that earlier seismic design procedures, where seismic forces calculated by elastic analysis procedures were reduced by a force reduction factor R, and were then added to the corresponding forces resulting from gravity effects, are fundamentally in error, since they grossly exaggerate the importance of the gravity effect, Priestley et al. (1996).

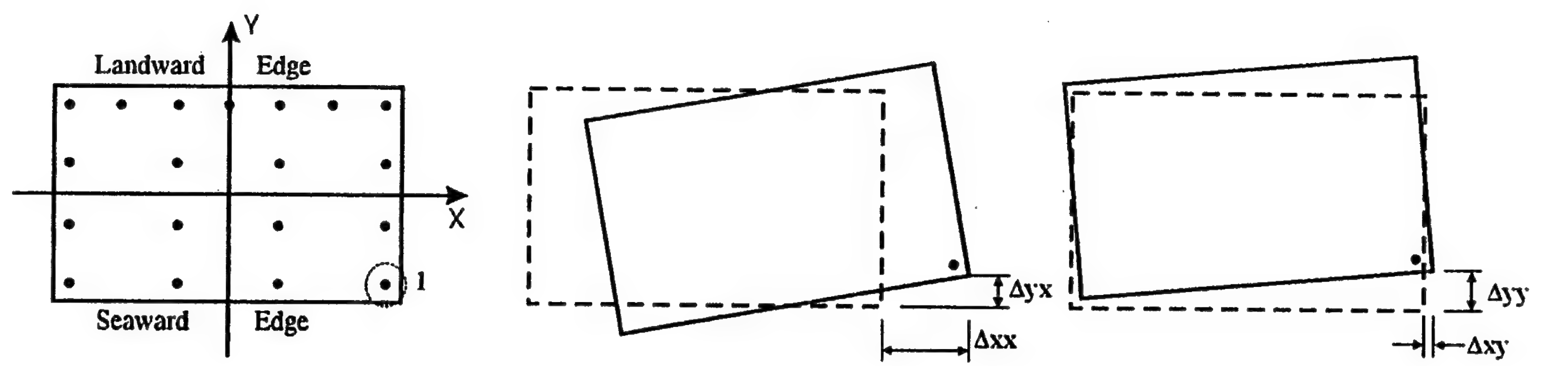
Combination of Orthogonal Effects

The traditional 100% X + 30%Y; 30% X + 100% Y combinations of orthogonal effects is retained. In this context the principal actions referred to are the displacements in the horizontal plane. This is clarified in Figure 3-19, where a plane view of a wharf is analyzed under both X and Y direction actions, resulting in the deflected shapes of Figure 3-19(a) and 3-19(b) respectively. Considering only the corner pile 1, the design displacements Δ_d corresponding to Equations 3-17 and 3-18 are given by:

$$\Delta_X = \Delta_{XX} + 0.3\Delta_{XY}$$

$$\Delta_Y = \Delta_{YX} + 0.3\Delta_{YY} \qquad (3\text{-}17)$$

$$\Delta_d = \sqrt{\Delta_X^2 + \Delta_Y^2}$$



(a) Plan view (b) Displaced shape, X excitation (c) Displaced shape, Y excitation

Figure 3-19 Plan view of wharf segment under X and Y seismic excitations.

$$\Delta_X = 0.3\Delta_{XX} + \Delta_{XY}$$

$$\Delta_Y = 0.3\Delta_{YX} + \Delta_{YY} \quad (3\text{-}18)$$

$$\Delta_d = \sqrt{\Delta_X^2 + \Delta_Y^2}$$

The displacements Δ_d corresponding to Equations 3-17 and 3-18 are then compared with the displacement capacities at the relevant performance limit state.

Methods Of Analysis For Seismic Response

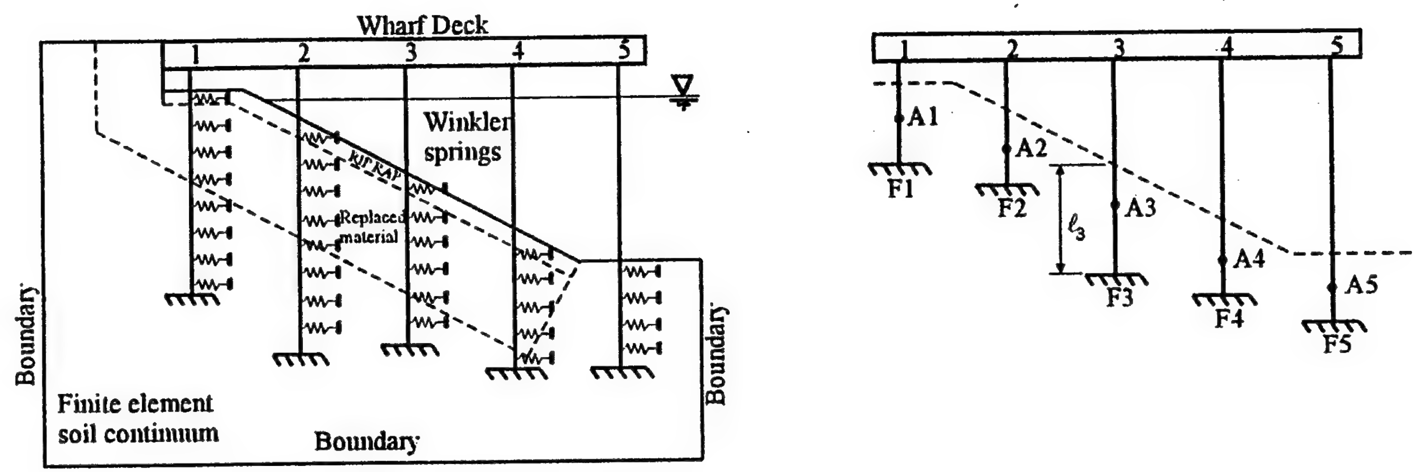
In comparison with many other structures such as multistory buildings, or multispan bridges, wharves and piers are frequently structurally rather simple. As such, analyses to determine seismic response can often utilize comparatively simple analysis procedures, as discussed below, without significant loss of accuracy. Complexity tends to come more from the high significance of soil-structure interaction, from significant torsional response resulting from the typical increase in effective pile lengths from landward to seaward sides (or ends) of the wharf (or pier), and from interaction between adjacent wharf or pier segments separated by movement joints with shear keys, rather than from structural configuration.

Analytical effort must be placed into the best possible modeling of these effects, and of the influence of cracking on member stiffness in the elastic range of response. Effort placed into the accurate modeling of member stiffness and soil-structure interaction effects will have a greater significance on the accurate prediction of peak response than will the choice of analytical procedure (e.g. Methods A, B, or D below). Indeed there is no point in carrying out a sophisticated time-history analysis unless detailed and accurate simulation of member stiffness and strength has preceded the analysis.

Modeling Aspects

Soil-Structure Interaction

Figure 3-20(a) represents a typical transverse section through a wharf supported on a soil foundation comprised of different materials (insitu sand and gravel, perhaps with clay lenses; rip-rap and other foundation improvement materials etc.). The most precise modeling of this situation would involve inelastic finite element modeling of the foundation material to a sufficient depth below, and to a sufficient distance on each side, such that strains in the foundation material at the boundaries would not be influenced by the response of the wharf structure. The foundation would be connected to the piles by inelastic Winkler springs at sufficiently close spacing so that adequate representation of the pile deformation relative to the foundation material, and precise definition of the in-



(a) Soil-Structure Continuum Model

(b) Equivalent-Depth-To-Fixed Model

Figure 3-20. Modeling soil-structure interaction for a wharf.

ground plastic hinging would be provided. Close to the ground surface, where soil spring stiffness has the greatest influence on structural response, the springs should have different stiffnesses and strengths in the seaward, landward, and longitudinal (parallel to shore) directions, as a consequence of the dyke slope.

The pile elements would be represented by inelastic properties based on moment-curvature analyses. Since the key properties of piles (namely strength and stiffness) depend on the axial load level, which varies during seismic response, sophisticated interaction modeling would be necessary.

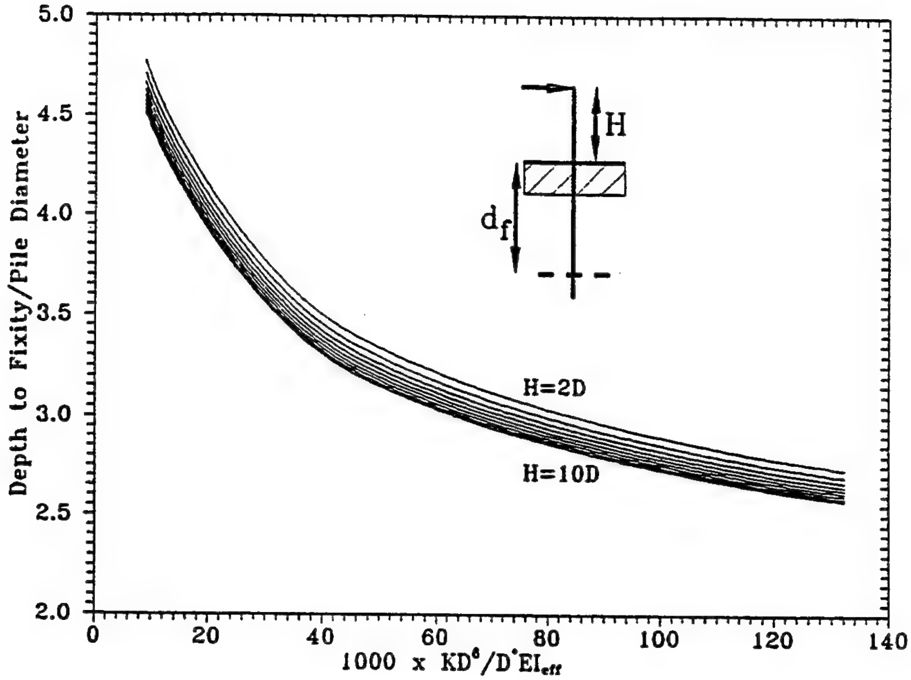
It is apparent that structural modeling of the accuracy and detail suggested above, though possible, is at the upper limit of engineering practice, and may be incompatible with the very considerable uncertainty associated with the seismic input. In many cases it is thus appropriate to adopt simplified modeling techniques. Two possibilities are illustrated in Figure 3-20. In Figure 3-20(a), the complexity may be reduced by assuming that the piles are fixed at their bases with the seismic input applied simultaneously and coherently to each pile base. The soil springs also connect the piles to the rigid boundary. Thus the assumption is made that the deformations within the soil are small compared with those of the wharf or pier.

In Figure 3-20(b), the complexity is further reduced by replacing the soil spring systems by shortening the piles to "equivalent fixity" piles, where the soil is not explicitly modeled, and the piles are considered to be fixed at a depth which results in the correct overall stiffness and displacement for the wharf or pier. Where different soil stiffnesses are appropriate for opposite directions of response, average values will be used, or two analyses carried out, based on the two different stiffnesses respectively. It should be noted that though this modeling can correctly predict stiffness, displacements and elastic periods, it will over-predict maximum pile in-ground moments. The model will also require adjustment for inelastic analyses, since in-ground hinges will form higher in the pile than the depth of equivalent fixity for displacements.

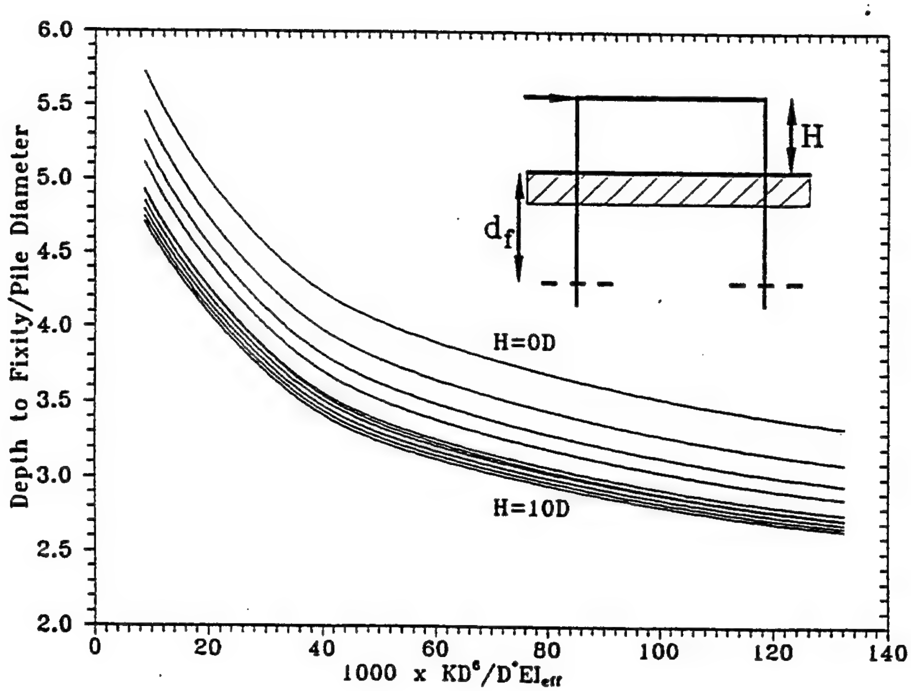
Geotechnical guidance will be needed to determine the appropriate depth to fixity (e.g. l_3 in Figure 3-20(b)), but an approximate value may be determined from the dimensionless charts of Figure 3-21. A typical value of $l_3 = 5D_p$ where D_p is the pile diameter, may be used as a starting point for design

Movement Joints

Long wharf structures, and less commonly piers, may be divided into segments by movement joints to facilitate thermal, creep and shrinkage movements. Typically the joints allow free longitudinal opening, but restrain transverse displacement by the incorporation of shear keys. Modeling these posses problems, particularly when elastic analysis methods are used. Under relative longitudinal displacement, the segments can open freely, but under closing displacements, high axial stiffness between the segments develops after the initial gap is closed. Relative transverse rotations are initially



(a) Free-head pile



(b) Fixed-head pile

Figure 3-21. Equivalent depth to fixity for CIDH piles.

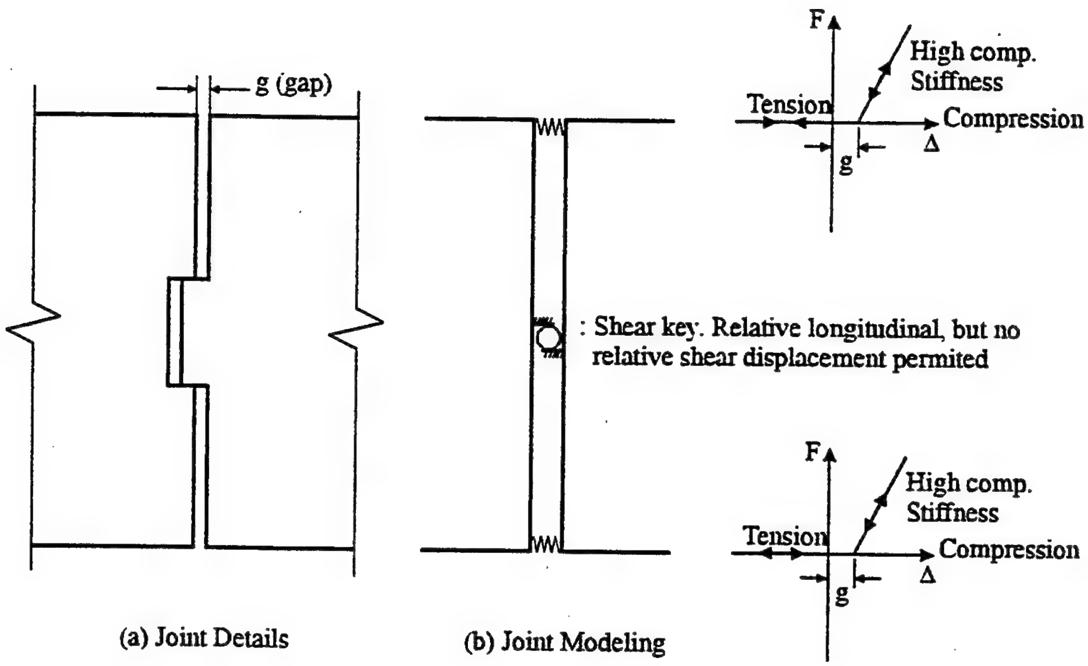


Figure 3-22. Modeling of movement joints.

unrestrained, but after rotations are sufficient to close one side of the joint, further rotation implies relative axial displacement, resulting in axial compression between the segments. The behavior can be modeled with a central shear spring allowing longitudinal displacement but restraining transverse displacements, and two axial springs, one at each end of the joint, as suggested in Figure 3-22. These two springs have zero stiffness under relative opening displacement, but have a very high compression stiffness after the initial gap Δ_g is closed.

Clearly it is not feasible to represent this nonlinear response in an elastic modal analysis. Even in an inelastic analysis accurate modeling is difficult because of the need to consider different possible initial gaps, and difficulty in modeling energy dissipation by plastic impact as the joint closes.

Method A: Equivalent Single Mode Analysis

Long wharves on regular ground tend to behave as simple one-degree-of-freedom structures under transverse response. The main complexity arises from torsional displacements under longitudinal response, and from interaction across shear keys between adjacent segments, as discussed above. An individual segment, particularly, will respond with significant torsional component when excited by motion parallel to the shore, or length of the wharf. Dynamic analyses, Priestley (1999) indicate that the torsional response is reduced when segments are connected by shear keys, and with multiple-segment wharves, the torsional response of the inner segments is negligible. It is thus conservative to estimate the displacement response based on the behavior of a single wharf segment between movement joints, except when calculating shear key forces.

Extensive inelastic time-history analyses of single and multi-segment wharves have indicated that an approximate upper bound on displacement response of the piles could be established by multiplying the displacement response calculated under pure transverse (perpendicular to the shore) excitation by a factor taking into account the orthogonal load combinations of Equations 3-17 and 3-18, including torsional components of response. For the critical shorter landward line of piles

$$\Delta_{\max} = \Delta_t \sqrt{1 + (0.3(1 + 20\ {}^{e}\!/_{L_L}))^2} \qquad (3\text{-}19)$$

where

Δ_t =displacement under pure translational response,
e = eccentricity between center of mass and center of rigidity
L_L =longitudinal length of the wharf segment.

Note that the corner piles of the line of piles at the sea edge of the wharf will be subjected to slightly higher than given by Equation 3-19, but these piles will not be

critical because of their greatly increased flexibility and hence increased displacement capacity, compared with the landward piles.

The shear force across shear keys connecting adjacent wharf segments cannot be directly estimated from a Method A analysis. However, time-history analysis of multiple wharf segments indicates that the highest shear forces will occur across movement joints in two-segment wharves, and that these forces decrease, relative to segment weight approximately in inverse proportion to wharf segment aspect ratio L_L/L_T where L_L and L_T where the longitudinal and transverse plan dimensions of the wharf segment pile group. The following expression may be used as an approximate upper bound to the shear key force V_{sk}:

$$V_{sk} = 1.5\ (e/L_L)\ V_{\Delta T} \qquad (3\text{-}20)$$

Where $V_{\Delta T}$ is the total segment lateral force found from a pushover analysis at the level of displacement Δ_T calculated for pure translational response at the appropriate limit state, L_L is the segment length, and e is the eccentricity between the center of stiffness and the center of mass.

Method A is of adequate accuracy for design of many simple structures when supplemented by a Method C pushover analysis, and is particularly useful for the preliminary stages of design, even of relatively important and complex structures. Final design verification may be made by one of the more sophisticated analysis methods.

The approximate lateral stiffness of a wharf structure analyzed using Method A should be determined from a pushover analysis, as discussed below in relation to Method C.

Method B Multi-Mode Spectral Analysis

Spectral modal analysis will be the most common method used for estimating maximum displacement levels, particularly when deck flexibility is significant. However, as noted above, when multiple segments of long wharves are connected by movement joints with shear keys, it is difficult to model the interactions occurring at the movement joints adequately, because of their non-linear nature.

It is thus doubtful if it is worth modeling the joint, particularly if conditions are relatively uniform along the wharf. It should also be recognized that it is unlikely that there will be coherency of input motion for different segments of a long wharf. Longitudinal motion of the wharf is likely to be reduced as a consequence of impact across joints, and restriction of resonant build-up.

It is thus reasonable, for regular wharf conditions to consider an analysis of single wharf segments between movement joints as "stand alone" elements. It will also

be reasonable in most cases to lump several piles together along a given line parallel to the shore an provide an analytical "super pile" with the composite properties of the tributary piles, to reduce the number of structural elements, which can be excessive, in a multi-mode analysis. Since the modal analysis will be used in conjunction with a Pushover Analysis, it will be worth considering adopting further reduction in the number of structural elements by representing the composite stiffness of a transverse line of piles, found from the Pushover analysis, by a single pile, and using a damping level appropriate to the expected displacement, is discussed in more detail in the following section.

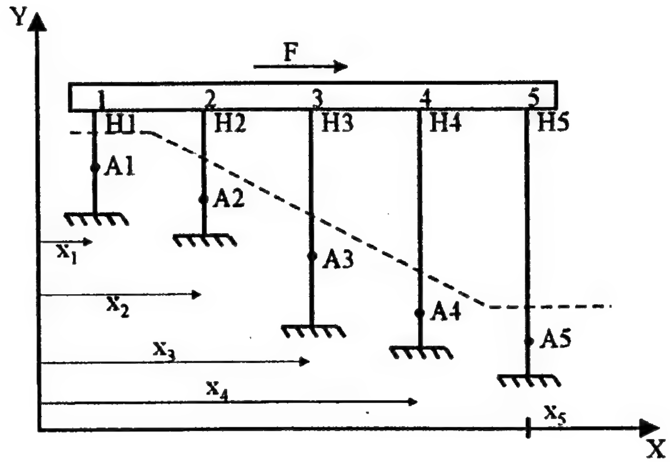
In most stand-alone analyses of single wharf or pier segments, there will be only three highly significant modes; two translational and one torsional. These will generally be closely coupled. As a consequence it will not generally be difficult to satisfy the 95% participating mass requirement, which is larger than commonly specified in design codes. Note that to correctly model the torsional response it is essential to model the torsional inertia of the deck mass. This can either be done by distributing the deck mass to a sufficiently large number of uniformly distributed mass locations, (with a minimum of 4 located at the radius of gyration of the deck area) or by use of a single mass point with rotational inertia directly specified. Deck mass should include a contribution for the mass of the piles. Typically adding 33% of the pile mass from deck level to point of equivalent foundation fixity is appropriate.

Because of the typical close coupling of the key modes of vibration, the modal responses should be combined using the Complete Quadratic Combination (CQC) rule, Wilson et al (1981).

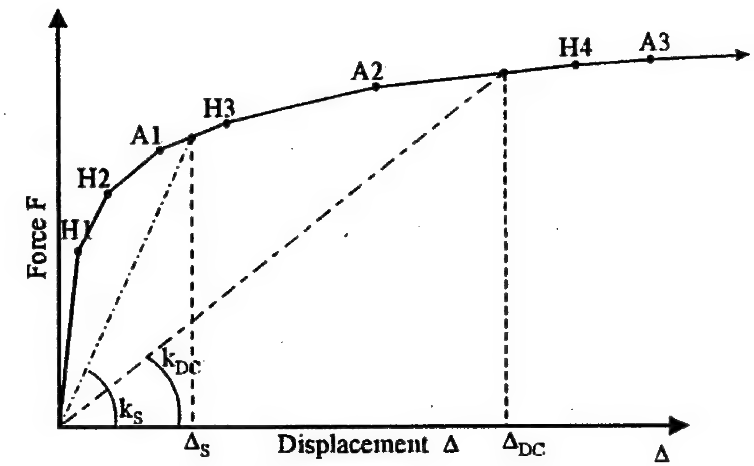
Method C: Pushover Analysis

Typically, as a result of large variations in effective depth to fixity of different piles, in a given wharf or pier, the onset of inelastic response will occur at greatly differing displacements for different piles. This is illustrated in Figure 3-23, representing the response of a 20-foot. segment of a typical wharf loaded in the transverse (perpendicular to shoreline) direction. There are a number of important consequences to this sequential hinging:

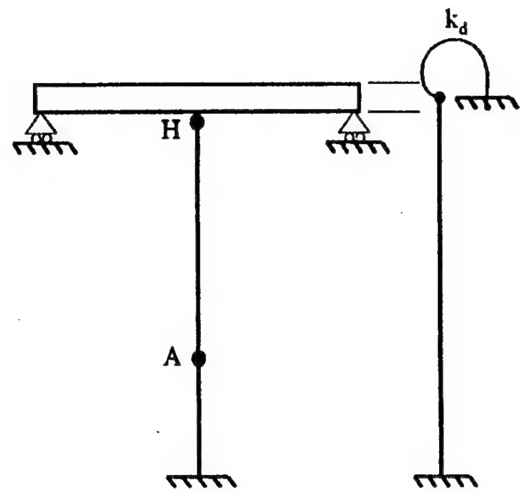
- It is difficult to adequately represent response by an elasto-plastic approximation, which is the basis of the common force-reduction factor approach to design.
- The appropriate elastic stiffness to be used in analysis is not obvious.
- Different piles will have greatly different levels of ductility demand, with the shortest piles being the most critical for design or assessment.
- The center of stiffness will move from a position close to the landward line of piles to a position closer to the center of mass as inelasticity starts to develop first in the landward piles.



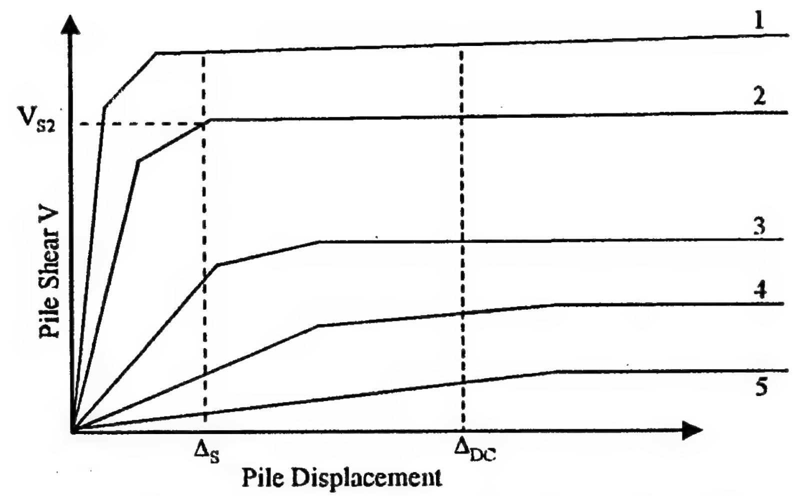
(a) Transverse section for analysis



(b) Force-Displacement Response and hinge sequence for Transverse Excitation.



(c) Pile Model for Longitudinal Analysis



(d) Individual Pile longitudinal Force-Displacement response

Figure 3-23. Pushover analysis of a wharf segment.

As a consequence it is not directly feasible to carry out an elastic analysis based on the typically assumed 5% equivalent viscous damping, and then determine member forces by reducing elastic force levels by a force-reduction factor. Amongst other failings, this will grossly underestimate the design forces for the longer piles. It is thus strongly recommended that a key aspect of the design or analysis process be a series of inelastic pushover analyses, in both transverse and longitudinal directions, where 2-D sections of the wharf or pier are subjected to incremental increases in displacement, allowing the inelastic force-displacement response (e.g. Figure 3-23(b)), the sequence of hinge formation, and the magnitude of inelastic rotation Θ_p developing in each hinge to be determined. The results of such analyses can be used to

- Determine an appropriate stiffness for a Method A or B analysis,
- Determine appropriate damping levels for elastic analyses, and
- Determine peak plastic rotations of critical hinges at the maximum displacements predicted by the Method A or B analyses.

These aspects are discussed in more detail in the following sections.

Elastic Stiffness from Pushover Analysis

It is recommended that elastic analyses used for Method A or B analyses be based on the substitute structure analysis approach, Gulkan and Sozen (1974), where the elastic characteristics are based on the effective stiffness to maximum displacement response anticipated for the given limit state, and a corresponding level of equivalent viscous damping, based on the hysteretic characteristics of the force displacement response (e.g. Figure 3-23(b). Thus for the transverse response, the stiffness for a 20ft. tributary length of wharf would be $\mathbf{k_S}$ or $\mathbf{k_{DC}}$ for the Serviceability and Damage Control limit states respectively, based on the expected limit states displacements Δ_S and Δ_{DC} respectively. Since these displacements will not be known prior to a Method A or B analysis, some iteration will be required to determine the appropriate stiffnesses.

For a Method A analysis, the transverse period can then be directly calculated from the stiffnesses and tributary mass, in the usual manner. For a Method B analysis, using a reduced number of piles as suggested above, longitudinal push analyses will be required on characteristic sections of wharf or pile. As suggested in Figure 3-23(c), the characteristic element for a wharf may be taken as a pile plus deck section extending midway to adjacent piles on either side. A set of pushover analyses for each of the characteristic piles A, B,...E, can then be carried out and plotted as shown in Figure 3-23(d). Again, based on the expected limit states displacements, the total stiffness of the tributary length of wharf considered can be calculated as

$$\mathbf{K_L} = \sum n_i V_i / \Delta_{ls} \qquad (3\text{-}21)$$

Where n_i is the number of piles in row i in the tributary width considered, and V_i is the pile shear force at the limit state deflection Δ_{ls}.

The center of rigidity, measured relative to the arbitrary datum shown in Figure 3-23(a) is given by

$$x_r = \frac{\sum(n_i V_i x_i)}{\sum(n_i V_i)} \qquad (3\text{-}22)$$

Thus, for a Method B analysis, the complete longitudinal and transverse stiffness of a given length of wharf may be represented by a single "super pile" located at the longitudinal center of the length modeled by the pile, and located transversely at the position defined by Equation 3-22, with stiffness values as defined above. Note that different stiffnesses and centers of rigidity will normally apply for the serviceability and damage control limit states.

Damping

The substitute structure approach models the inelastic characteristics of structures by elastic stiffness and damping levels appropriate for the maximum response displacements, rather than using the initial elastic parameters. The advantage is a more realistic representation of peak response, and an elimination of the need to invoke force-reduction factors to bring member force levels down from unrealistically high elastic values to realistic levels. The damping level is found from the shape of the complete hysteresis loop for a single cycle of displacement at maximum response as illustrated in Figure 3-24. In this, the skeleton force-displacement curve for both directions of response is calculated by a pushover analysis, as illustrated in Figure 3-23(b). The unloading curve is based on a modified Takeda approach where the unloading stiffness k_u is related to the structure ductility μ, defined in Figure 3-24, and the initial stiffness k_I by

$$k_u = k_I \ \mu^{-1/2} \qquad (3\text{-}23)$$

The residual displacement Δ_r is thus given by

$$\Delta_r = (\Delta_{ls} - F_m/k_u) \qquad (3\text{-}24)$$

The remainder of the stabilized hysteresis loop is constructed as shown in Figure 3-24 . The equivalent viscous damping, as a percentage of critical damping, is then given by

$$\eta = A_h / (2\pi . F_m \Delta_{ls}) \ 100\% \qquad (3\text{-}25)$$

where A_h is the area of the stabilized loop, shown shaded in Figure 3-24.

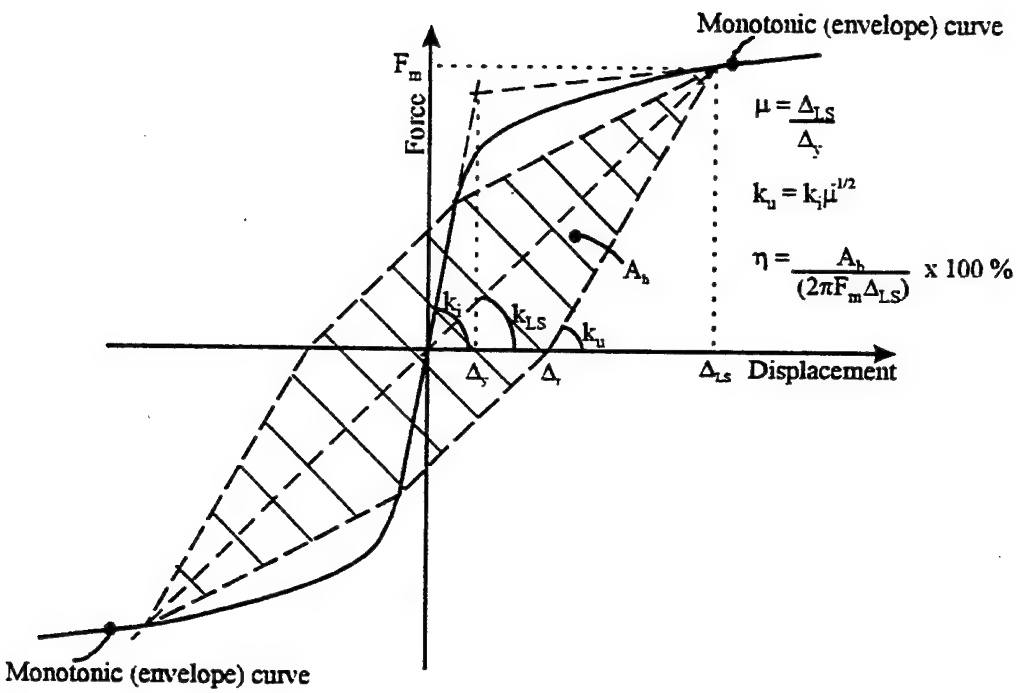


Figure 3-24. Equivalent viscous damping for substitute-structure analysis.

For initial analyses of wharves or piers supported on reinforced or prestressed concrete piles, damping values of 10% and 20% at the serviceability and damage control limit states will generally be appropriate.

Capacity Design Checks

When the pushover analyses are based on the dependable material propertied for the wharf or pier elements, the member forces resulting from the analyses represent lower bound estimates of the forces developed at the appropriate limit state. Although this may be appropriate for determining required strength of plastic hinge regions, and for ensuring that calculated limit state displacements are not exceeded, the results will not be appropriate for determining the required strength of members whose force levels must not exceed there dependable capacities during seismic response. It is clear that if material strengths exceed the specified strength, then the flexural strength of plastic hinges may significantly exceed the dependable strength, and as a consequence, the shear forces in piles and deck members may be as much as 20-30% higher than predicted by the pushover analysis.

In order to ensure that a conservative upper limit on the possible strength of members or actions which are to be protected against inelastic response is achieved, a second set of pushover analyses should be carried out, where the material strengths adopted represent probable upper bounds. This approach, based on Capacity Design Principles, is discussed in detail in Priestley et al. (1996).

Iteration Considerations

When checking a completed design, iteration will be needed to ensure compatibility between the Method A or B elastic analysis used to determine response displacements, and the stiffness and damping values determined from the pushover analysis. Typically the necessary adjustments to the substitute structure characteristics take very few cycles to converge with adequate accuracy.

In the design process, the limit state displacements will generally be known before the design strength is established, provided the type and diameter of pile is known. This is discussed further under structural response. In this case the iteration required will be centered on determining the correct number and location of piles to limit the response displacements to the structural limit state displacements.

Method D : Inelastic Time-History Analysis

Inelastic time-history analysis is potentially the most accurate method for estimating the full seismic response of a wharf or pier. It has the capability of

determining the maximum displacements, and the inelastic rotations in plastic hinges, from a single analysis. However, to do this, it is necessary to model each pile individually, and to simulate the pile/soil interaction by a series of Winkler springs as discussed above. This can result in unacceptable matrix sizes for analysis of wharves or piers with very large numbers of piles. An alternative is to combine the analysis with inelastic push analyses, and represent groups of piles by equivalent "super piles" as discussed above for modal analysis. Although this somewhat reduces the attraction of the time-history analysis, it makes the analysis more tractable, and still enables several advantages of the method, not available with Methods A or B, to be retained.

A disadvantage of the method is that considerable variation in response can be obtained between two different spectrum-compatible acceleration records. As a consequence it is essential to run an adequately large number of simulations using different acceleration records, and to average the results. A minimum of five spectrum-compatible records is recommended.

The following points need to be considered before undertaking a time-history analysis:

1. A full simulation of the wharf or pier will require use of a computer program capable of modeling 3-D response. There are comparatively few 3D inelastic time-history programs available at present (March 1999), and experience with them outside of research applications is rather limited. Frequently approximations must be made relating to hysteresis rules and strength interactions in orthogonal directions which make the added sophistication of time-history analysis of reduced utility.
2. If the deck has sufficient rigidity to justify its approximation as a rigid element both in-plane and out-of-plane, a 2-D plan simulation may provide adequate accuracy. However, special multi-direction spring elements with realistic hysteresis rules are required for such a simulation.
3. Time-history analysis enables different stiffnesses to be used for different directions of response. Thus, provided the necessary hysteresis rules are available, it will generally be possible to model the higher stiffness for movement into the shore than for movement away from the shore, and to have a separate stiffness for longitudinal response.
4. It is comparatively straightforward to model the interactions across shear keys, using inelastic time-history analysis.
5. When modeling reinforced or prestressed concrete members, degrading stiffness models such as the Modified Takeda rule should be adopted. There is little point in carrying out time history analyses if simplified rules, such as elasto-plastic, or even bilinear stiffness rules are adopted.
6. Care needs to be exercised into how elastic damping is handled. It is common to specify 5% elastic initial stiffness related damping. This can greatly overestimate the damping at high ductilities. In fact, since the hysteretic rules available in the literature have generally been calibrated to experimental results, the justification for adding elastic damping, which should be apparent in the experimental hysteresis loops, is of doubtful validity. There is thus a case for ignoring elastic damping, if

levels of inelastic response are high. However, at low levels of displacement response, the simplifications inherent in the hysteresis rules generally mean that the damping is underestimated for "elastic" or near-elastic response.

7. Results from a time-history analysis should always be compared with results from a simplified approach (e.g. a Method A or B analysis) to ensure that reasonable results are being obtained.

Method E: Gross Foundation Deformation Analysis

As discussed above, the assumption will normally be made that the foundation material is competent when carrying out any of the analyses described above. However, examination of the performance of wharves and piers in past earthquakes reveals that liquefaction and foundation sliding or slumping are common. Analysis techniques to estimate the sensitivity of the foundation to such failures, and the extent of deformation to be expected, are dealt with elsewhere in this document. Where geotechnical analyses indicate that moderate permanent deformations are to be expected, the structure should be analyzed under the deformed soil profile to estimate the influence on the structure. This will generally require at least a 2-D analysis incorporating discrete modeling of piles and soil springs, with foundation deformations applied at the boundary ends of the springs, as appropriate.

Structural deformations resulting from dynamic vibratory response, and estimated by any of methods A to D will generally occur much earlier in the response to an earthquake than will gross soil deformations. Indeed, liquefaction failure may well occur some minutes after the ground motion has ceased. Consequently it is not necessary to combine the results from a method E analysis with the results of a method A to D analysis. It will be sufficient to confirm that response of each type is individually satisfactory.

Structural Criteria For Piers And Wharves

Deformation Capacity Of Pile Plastic Hinges

The ability of wharves and piers to respond inelastically to seismic excitation depends on the displacement capacity of pile plastic hinges. This displacement capacity will depend on what type of piles are used in the structure, their length, cross-section dimensions, axial load, and material properties. A brief discussion of the deformation capacities of different pile types is included below. This is based on an examination of their moment-curvature characteristics.

Until recently, piles were designed neglecting the confinement effects of the reinforcing on the concrete. Work by Joen and Park (1990a) shows the significant increase in moment capacity by considering the effect of spiral confinement on the concrete. This work uses the Mander et al (1988) concrete model for confined and

unconfined concrete. Typically the ultimate compressive concrete strain of unconfined concrete is about 0.003 for use in computing flexural strength as reported by Priestley et al. (1992). For confined concrete the following may be used, Priestley et al. (1992),

$$\varepsilon_{cu} = 0.004 + (1.4\, \rho_s\, f_{yh}\, \varepsilon_{sm}) / f'_{cc} \geq 0.005 \qquad (3\text{-}26)$$

where

ρ_s	effective volume ratio of confining steel
f_{yh}	yield stress of confining steel
ε_{sm}	Strain at peak stress of confining reinforcement, 0.15 for grade 40 and 0.12 for grade 60
f'_{cc}	Confined strength of concrete approximated by 1.5 f_c'

Moment-Curvature Characteristics of Piles.

The key tool for investigating the deformation characteristics of piles is moment-curvature analysis. The analysis adopted must be capable on modeling the full stress strain curves of reinforcement and concrete realistically. For the concrete, this entails discrimination between the behavior of unconfined cover concrete, if present, and that of the confined core, which will have enhanced strength, and particularly enhanced deformation capacity. Elastic, yield plateau, and strain-hardening sections of the steel stress-strain curve should be modeled separately; a simple elasto-plastic or bi-linear representation will be inadequate for mild steel reinforcement, but may be adequate for prestressing steel. The computer code used should provide output of moment and curvature at regular intervals of a sequential analysis to failure, and should identify the peak values of extreme fiber concrete strain, and maximum reinforcement and/or prestressing strain at each increment of the analysis, so that curvatures corresponding to the serviceability and damage-control limit state can be identified.

A sample moment-curvature response, appropriate for a reinforcing steel dowel connection between a prestressed pile and a reinforced concrete deck is shown in Figure 3-25. For structural analysis, it will often be adequate to represent the response by a bi-linear approximation, as shown. The initial elastic portion passes through the first yield point (defined by mild steel yield strain, or an extreme fiber concrete compression strain of 0.002, whichever occurs first), and is extrapolated to the nominal flexural strength. This defines the nominal yield curvature, ϕ_y . The second slope of the bi-linear approximation joins the yield curvature and the point on the curve corresponding to the damage-control limit-state strains. As defined in the document, these are taken to be:

Concrete extreme fiber compression strain:

Pile/deck hinge: Value given by Equation 3-26, but <0.025

In-ground hinge:	Value given by Equation 3-26, but <0.008
Reinforcing steel tension strain:	0.05
Structural Steel (pile and concrete filled pipe):	0.035
Prestressing strand:	
Pile/deck hinge:	0.04
In-ground hinge:	0.015
Hollow steel pipe pile	0.025

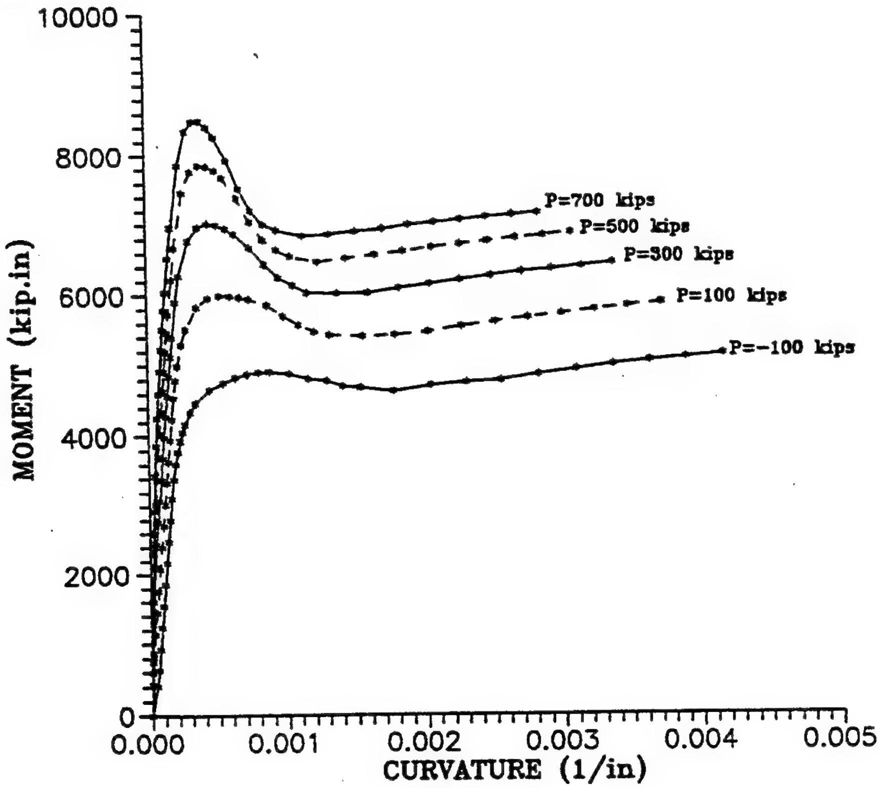
Equation 3-26 defines a safe lower bound estimate of the ultimate compression strain of concrete confined by hoops or spirals, Priestley et al. (1996). Actual compression failure, initiated by fracture of spiral or hoop confinement will typically not occur until strains are on average 50% larger than the value given by Equation 3-26, providing an adequate margin for uncertainty of input.

Curvatures at the serviceability limit state are based on strain limits sufficiently low so that spalling of cover concrete will not occur under the Level 1 earthquake, and any residual cracks will be fine enough so that remedial grouting will not be needed. As defined in the document, these are taken to be:

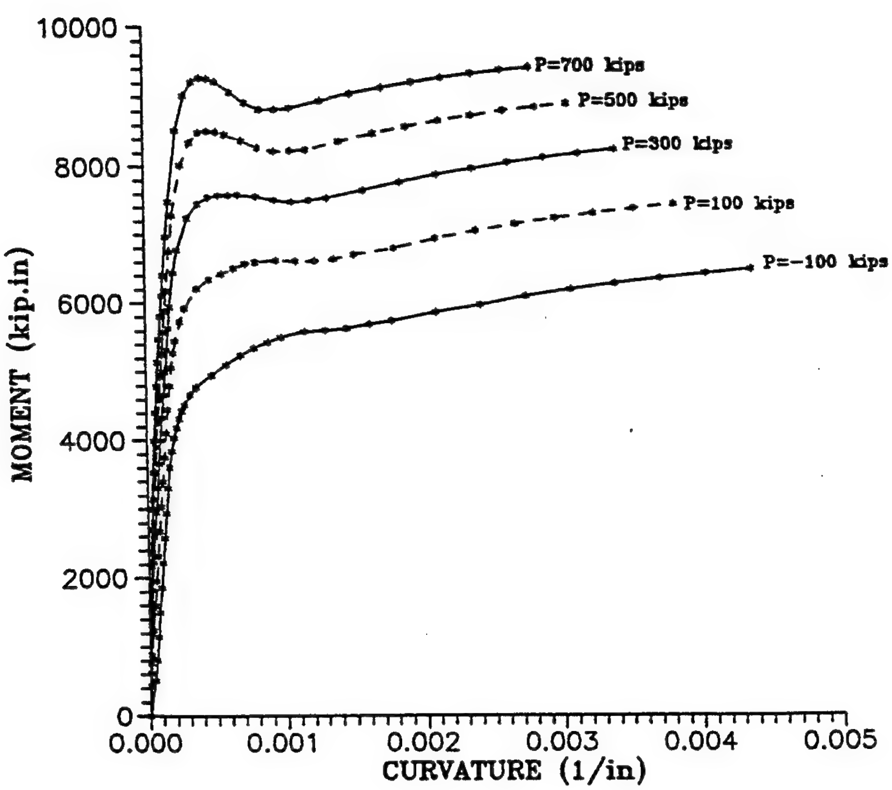
Concrete extreme fiber compression strain:	0.004
Reinforcing steel tension strain:	0.010
Prestressing strand incremental strain:	0.005
Structural steel (pile and concrete-filled pipe):	0.008
Hollow steel pipe pile:	0.008

The specified incremental strain for prestressing strand is less than for reinforcing steel to ensure that no significant loss of effective prestress force occurs at the serviceability limit state. For steel shell piles, either hollow or concrete filled, it is recommended that the maximum tension strain in the shell be also limited to 0.01 to ensure that residual displacements are negligible. Note that if the steel shell pile is connected to the deck by a dowel reinforcing bar detail, the connection is essentially a reinforced concrete connection confined by the steel shell. As such the limit strains for reinforced concrete apply.

Figure 3-26 shows examples of theoretical moment-curvature curves for different types of piles, each with an outside diameter of 610mm, and each subjected to different levels of axial load. Comparison of the curves for the two reinforced concrete doweled connections of Figures 3-26(a) and 3-26(b), show the significant influence of the

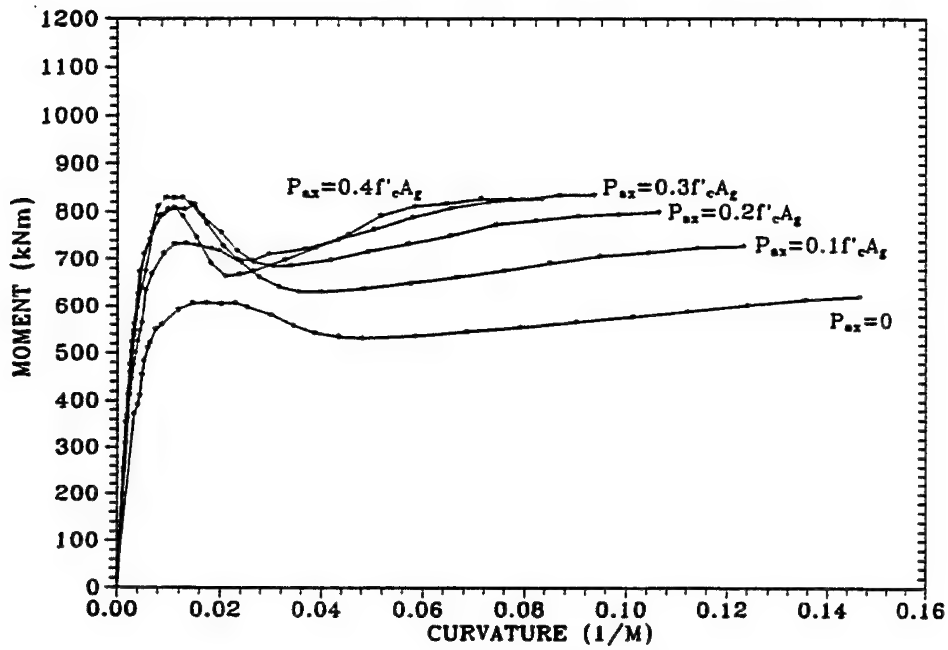


a) cover = 4 inches (102mm) RC dowel connection 8#10 bars

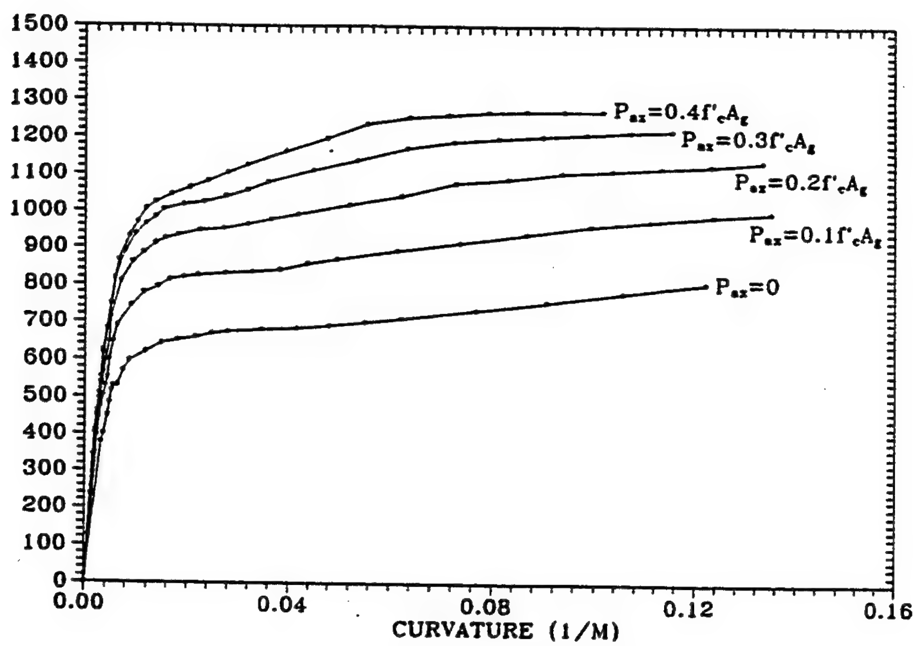


b) cover = 2.5 inches (64mm) RC dowel connection 8#10 bars

Figure 3-26. Moment-curvature curves for 610 mm diameter piles.

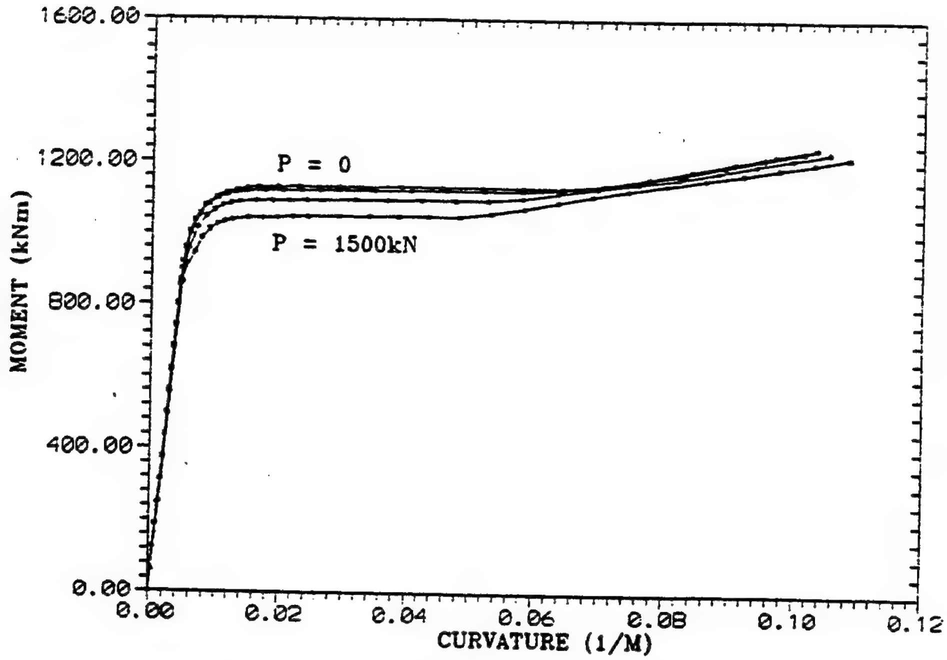


c) prestressed pile, F/A = 6.4 Mpa, cover = 76mm

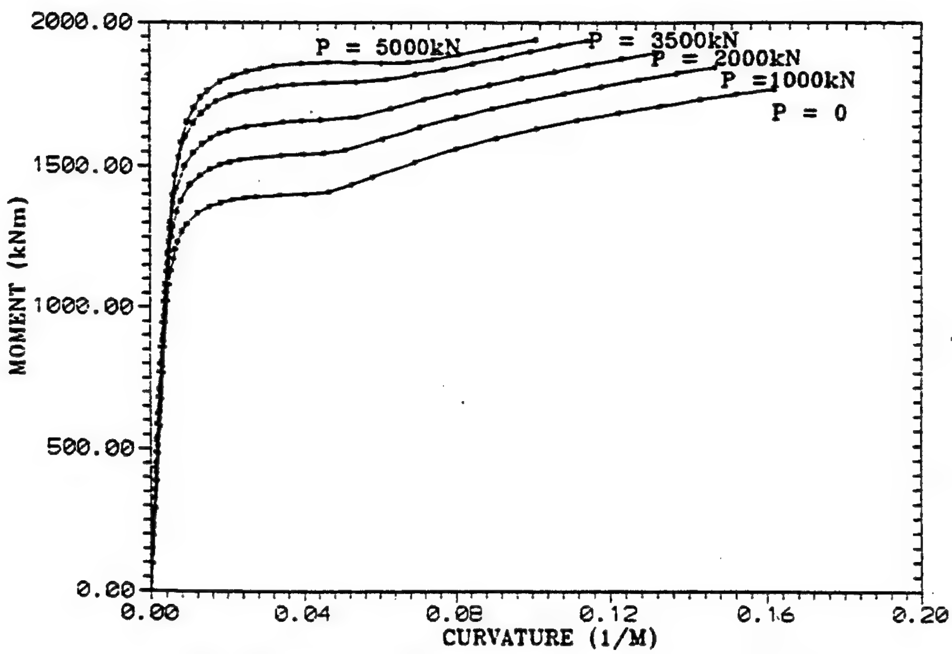


d) prestressed pile, F/A = 6.4 Mpa, cover = 25mm

Figure 3-26. Continued.



e) Steel shell pile, t = 12mm



f) Concrete filled steel shell pile, t = 12mm

Figure 26. Continued.

concrete cover on the moment-curvature characteristics. Flexural strength is significantly higher with the reduced concrete cover of Figure 3-26(b). Sections with high cover (102mm) show significant reductions in moment capacity when spalling initiates, for all levels of axial load, while piles with 63.5mm cover exhibit much more satisfactory response. Similar behavior is apparent for circular or octagonal prestressed piles, as shown for two different values for concrete cover in Figures. 3-26(c) and 3-26(d). Moment-curvature curves for hollow and concrete-filled steel shell piles are shown in Figures 3-26(e) and 3-26(f). For the hollow steel shell pile, the influence of axial load is rather limited, with axial compression tending to reduce the flexural strength. The influence of internal concrete is to increase the moment capacity significantly, particularly when axial load is high. Another advantage of the concrete infill is that it reduced the sensitivity of the steel shell to local buckling.

Rectangular section reinforced or prestressed concrete piles confined by square spirals and a rectangular distribution of longitudinal reinforcing bars or prestressing strand should not be used for new construction, since the confinement provided to the core concrete by the rectangular spirals is of very low efficiency. For assessment of existing structures, the concrete core maximum strain corresponding to the damage-control limit state should not be taken larger than 0.007 at the pile/deck hinge. Higher maximum concrete strains , as given by Equation 3-26, are appropriate if the core of the rectangular pile is confined by a circular spiral, and the longitudinal reinforcement or prestressing is also circularly disposed. However, such sections typically have low ratios of concrete core area to gross section area, resulting in flexural strength of the confined core being less than that of the unconfined gross section.

Hollow prestressed piles are sometimes used for marine structures. These, however, have a tendency to implode when longitudinal compression strains at the inside surface exceed 0.005. Consequently the damage control limit state should have an additional requirement to the strain limits defined in the document, that inside surface compression strains must not exceed 0.005. Note that to check for this condition, the moment-curvature analysis must be able to model spalling of the outside cover concrete when strains exceed about 0.004 or 0.005. The outside spalling can cause a sudden shift of the neutral axis towards the center of the section, resulting in strains on the inside surface reaching the critical level soon after initiation of outside surface spalling. More information on piles is available in Priestley and Seible (1997).

Elastic Stiffness

The effective elastic stiffness may be calculated from the slope of the "elastic" portion of the bi-linear approximation to the moment-curvature curve (e.g. Figure 3-25), as

$$EI_{eff} = M_N / \phi_y \qquad (3\text{-}27)$$

The value of the yield curvature, ϕ_y , for a reinforced concrete pile, or pile/deck connection is rather insensitive to axial load level or longitudinal reinforcement ratio. Results of analyses of a large number of cases indicate that Equation 3-27 may be approximately expressed as a fraction of the gross section stiffness as

$$EI_{eff} / EI_{gross} = 0.3 + N / (f'_c . A_{gross}) \qquad (3\text{-}28)$$

Where N is the axial load level, and A_{gross} is the uncracked section area.

For prestressed piles the effective stiffness is higher than for reinforced concrete piles, and values in the range $0.6 < EI_{eff} / EI_{gross} < 0.75$ are appropriate. For prestressed piles with reinforced dowel connections to the deck, the effective stiffness should be an average of that for a reinforced and a prestressed connection, or a short length, approximately $2D_p$ long of reduced stiffness appropriate for reinforced pile should be located at the top of the prestressed pile.

Plastic rotation

The plastic rotation capacity of a plastic hinge at a given limit state depends on the yield curvature, ϕ_y, the limit-state curvature,(ϕ_S or ϕ_{LS}) and the plastic hinge length L_p, and is given by

$$\Theta_p = \phi_p L_p = ((\phi_S \text{ or } \phi_{LS}) - \phi_y)\, L_p \qquad (3\text{-}29)$$

Plastic Hinge Length

The plastic hinge length for piles depends on whether the hinge is located at the pile/deck interface, or is an in-ground hinge. Because of the reduced moment gradient in the vicinity of the in-ground hinge, the plastic hinge length is significantly longer there. For pile/deck hinge locations with reinforced concrete details, the plastic hinge length can be approximated by

SI units $\quad L_P = 0.08\, L + 0.022 f_y d_b > 0.044 f_y d_b$ (Mpa, mm) $\qquad$ (3-30a)

US units $\quad L_P = 0.08\, L + 0.15 f_y d_b > 0.30 f_y d_b$ (ksi, in.) $\qquad$ (3-30b)

where f_y is the yield strength of the dowel reinforcement, of diameter d_b, and L is the distance from the pile/deck intersection to the point of contraflexure in the pile.

For prestressed piles where the solid pile is embedded in the deck (an unusual detail in the USA), the plastic hinge length at the pile/deck interface can be taken as

$$L_p = 0.5 D_p \qquad (3\text{-}31)$$

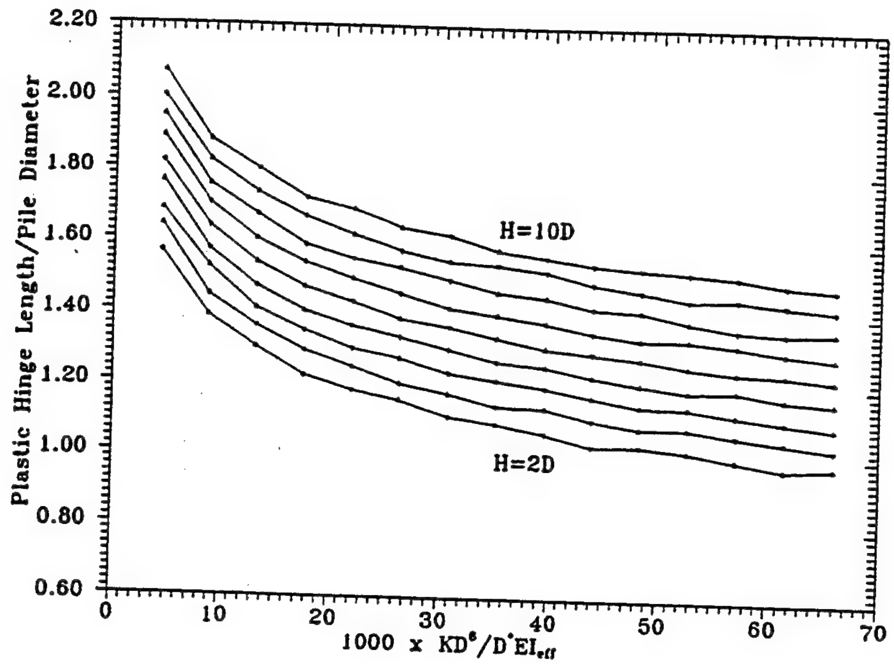


Figure 3-27. In-ground plastic hinge length
(H = height of contraflexure point above ground, D = pile diameter)

For in-ground hinges, the plastic hinge length depends on the relative stiffness of the pile and the foundation material. The curves of Figure 3-27 relate the plastic hinge length of the in-ground hinge to the pile diameter, D_p, a reference diameter D* = 1.82m, and the dimensionless soil lateral subgrade coefficient, k (N/m^3).

For structural steel sections, and for hollow or concrete-filled steel pipe piles, the plastic hinge length depends on the section shape, and the slope of the moment diagram in the vicinity of the plastic hinge, and should be calibrated by integration of the section moment-curvature curve. For plastic hinges forming in steel piles at the deck/pile interface, and where the hinge forms in the steel section rather than in a special connection detail (such as a reinforced concrete dowel connection), allowance should be made for strain penetration into the pile cap. In the absence of experimental data, the increase in plastic hinge length due to strain penetration may be taken as 0.25 D_p, where D_p is the pile diameter or pile depth in the direction of the applied shear force.

In-Ground Hinge Location

The location of the in-ground plastic hinge for a pile may be found directly from an analyses where the pile is modeled as a series of inelastic beam elements, and the soil is modeled by inelastic Winkler springs. When the pile/soil interaction is modeled by equivalent-depth-to-fixity piles, the location of the in-ground hinge is significantly higher than the depth to effective fixity, as illustrated in Figure 3-28 by the difference between points A, at the effective fixity location, and B, the location of maximum moment. Note that when significant inelastic rotation is expected at the in-ground hinge, the location of B tends to migrate upwards to a point somewhat higher than predicted by a purely elastic analysis. It is thus important that its location, which is typically about 1- 2 pile diameters below grade, should be determined by inelastic analysis. An alternative is to determine the depth of B using the dimensionless curves of Figure 3-29, which uses the same dimensionless parameters as Figure 3-27.

Pile Force-Displacement Response

The information provided in the previous few section enables an inelastic force displacement response to be developed individually for each pile. This may be directly carried out on a full 2-D section through the wharf, involving many piles, as part of a push-over analysis, or it may be on a pile-by-pile basis, with the push-over analysis assembled from the combined response of the individual piles. With respect to the equivalent-depth-to-fixity model of Figure 3-28, the pile is initially represented by an elastic member, length L, with stiffness EI_{eff} given by Equation 3-27 or 3-28, as appropriate, and the deck stiffness represented by a spring k_d as shown. Often it will be sufficiently accurate to assume the deck to be flexurally rigid, particularly with longer piles.

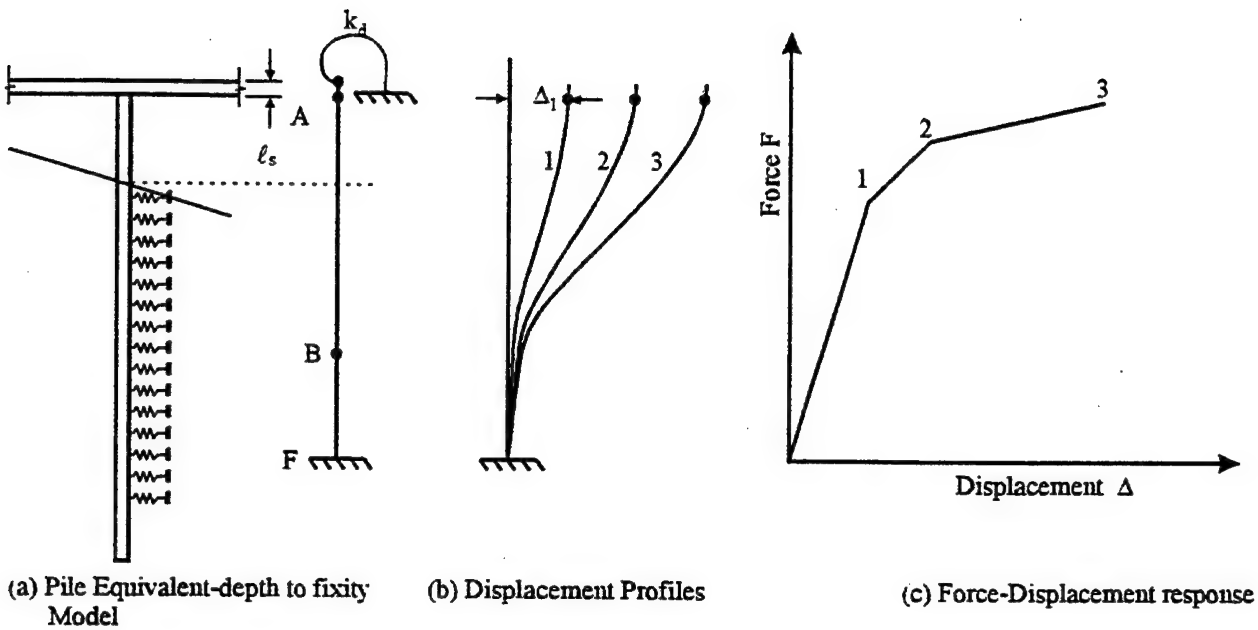
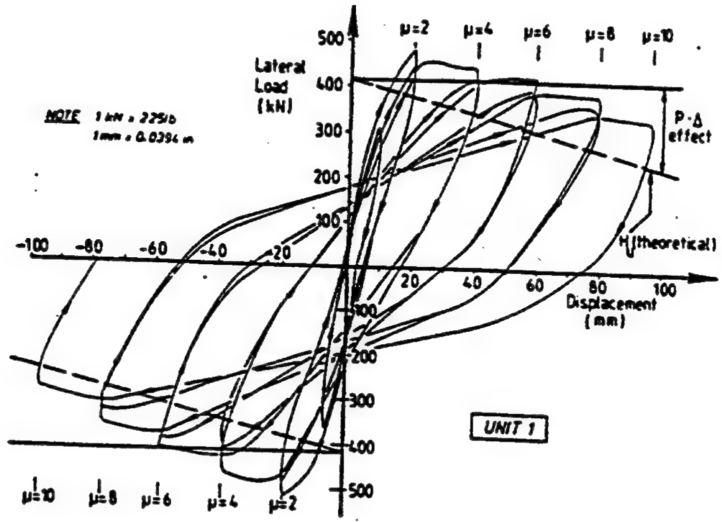
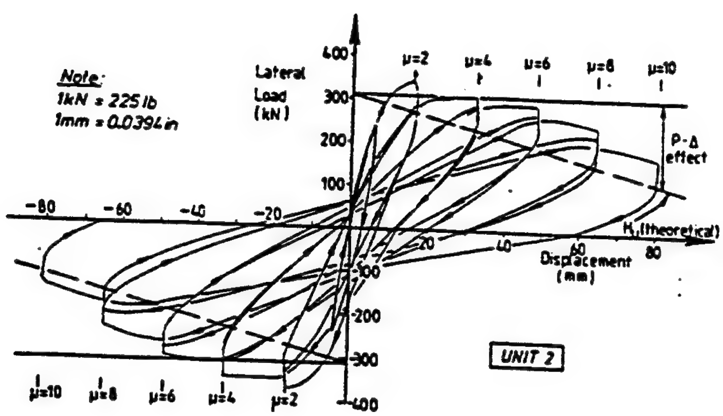


Figure 3-28. Force – displacement response of an isolated pile.



(a) Fully confined pile with additional longitudinal mild steel reinforcement.



(b) Fully confined pile without additional longitudinal mild steel reinforcement.

Figure 3-29. Load –displacement hysteresis loops for prestressed piles tested by Falconer and Park.

The deflection and force corresponding to development of nominal strength M_n at the pile/deck hinge can then be calculated. Note that for elastic deformation calculations the interface between the deck and pile should not be considered rigid. The effective top of the pile should be located a distance $0.022f_y d_b$ into the deck, to account for strain penetration. This is particularly important for short piles. This additional length applies only to displacements – maximum moment should still be considered to develop at the soffit of the deck.

The elastic calculations above result in a pile displacement profile marked 1 in Figure 3-28(b), and the corresponding point on the force displacement curve of Figure 3-28(c). For the next step in the pile pushover analysis, an additional spring k_{pt} must be added at A, the deck/pile interface (i.e. the deck soffit) to represent the inelastic stiffness of the top plastic hinge. This stiffness can be determined from Figure 3-25 as

$$k_p = \frac{(M_u - M_N)}{(\phi_{LS} - \phi_y)L_p} \quad . \qquad (3\text{-}32)$$

Essentially this spring is in series with the deck spring. Additional force can be applied to the modified structure until the incremental moment at B is sufficient to develop the nominal moment capacity at the in-ground hinge. The deflection profiles and force-displacement points marked 2 in Figures. 3-28 (b) and (c) refer to the status at the end of this increment. Finally the modified structure with plastic hinges at the deck/pile interface, and B is subjected to additional displacement until the limit state curvature ϕ_{LS} is developed at the critical hinge, which will normally be the deck/pile hinge. Note that the inelastic spring stiffnesses at the two hinge locations will normally be different, due to differences in the structural details between the in-ground and hinge locations, and due to the different plastic hinge lengths.

The procedure outlined above is sufficiently simple to be carried out by spreadsheet operations, and has the advantage that the post yield moment-rotation stiffness of the hinges can be accurately modeled. This function is not available in all push-over codes.

Material Properties for Plastic Hinges

For both design of new structures, and the assessment of existing structures, it is recommended that the moment-curvature characteristics of piles be determined based on probable lower bound estimates of constituent material strengths. This is because strength is less important to successful seismic resistance than is displacement capacity. For the same reason, there is little value in incorporating flexural strength reduction factors in the estimation of the strength of plastic hinges. Flexural strength reduction factors were developed for gravity load design, where it is essential to maintain a margin of strength over loads to avoid catastrophic failure. Inelastic seismic response, however, requires that the flexural strength be developed in the design level earthquake.

Incorporation of a strength reduction factor will not change this, and at best may slightly reduce the ductility demand. In fact, it has been shown, Priestley (1997) that even this is doubtful, and the provision of extra strength, resulting from the incorporation of strength reduction factors in the design of plastic hinges, may reduce the ductility capacity slightly, compensating for any benefits accruing from additional strength.

On the other hand, the use of nominal material strengths and strength reduction factors in design or assessment will place a demand for corresponding increases in the required strength of capacity protected actions, such as shear strength. This is because the maximum feasible flexural strength of plastic hinges, which dictates the required dependable shear strength will be increased when design is based on conservatively low estimates of material strength. This will have an adverse economic impact on the design of new structures, and may result in an unwarranted negative assessment of existing structures.

As a consequence, the following recommendations Priestley et al. (1996) are made for the determination of moment-curvature characteristics of piles.

Concrete compression strength	$f'_{ce} = 1.3f'_c$
Reinforcement Yield strength	$f_{ye} = 1.1f_y$
Prestress strand ultimate strength	$f_{pue} = 1.0f_{pu}$

Where f'_c is the specified 28 day compression strength for the concrete, and f_y and f_{pu} are the nominal yield and ultimate strength of the mild steel reinforcement and prestressing strand respectively. For assessment of existing structures, higher concrete compression strength will often be appropriate due to natural aging, but should be confirmed by in-situ testing.

In both new design and assessment of existing structures, the flexural strength reduction factor for pile plastic hinges should be taken as unity.

Confinement of Pile Plastic Hinges

Research by Budek et al (1997) has shown that lateral soil pressure at the in-ground plastic hinge location helps to confine both core and cover concrete. This research found that as a consequence of this confinement, for both reinforced and prestressed circular piles, the plastic rotation capacity of the in-ground hinge was essentially independent of the volumetric ratio of confinement provided. As a consequence of this, and also as a result of the low material strains permitted in the in-ground hinge at the damage-control limit state, much lower confinement ratios are possible than those frequently provided for piles. It is recommended that, unless higher confinement ratios are required for pile-driving, the confinement ratio for the in-ground portion of the pile need not exceed

$$\rho_s = \frac{4A_{sp}}{(D_p - 2c_o)s} = 0.008 \qquad (3\text{-}33)$$

where A_{sp} = area of the spiral or hoop bar,
D_p = pile diameter
c_o = concrete cover to center of hoop or spiral bar
s = spacing of spiral or hoop along the pile axis.

In the vicinity of the potential plastic hinge at the top of the pile, the amount of spiral or hoop reinforcement can be adjusted to ensure that the ultimate compression strain given by Equation 3-26 is adequate to provide the required displacements at the damage-control limit state. Thus the designer has some ability to optimize the design of pile confinement, dependent on the displacement requirement predicted under the lateral response analysis. The calculated value should be supplied for at least 2 D_p from the critical section. Because of uncertainties associated with final position of the tip of a driven pile prior to driving, a longer region of pile should have the increased confinement determined from the above approach. In many cases the full pile length will conservatively be confined with the volumetric ratio required at the deck/pile hinge location.

As an alternative to the approach outlined above, a prescriptive requirement, modified from bridge design, and defined by Equation 3-34 may be adopted:

$$\rho_s = 0.16\frac{f'_{ce}}{f_{ye}}(0.5 + \frac{1.25(N_u + F_p)}{f'_{ce}A_{gross}}) + 0.13(\rho_l - 0.01) \qquad (3\text{-}34)$$

where N_u = axial compression load on pile, including seismic load,
F_p = axial prestress force in pile
ρ_l = longitudinal reinforcement ratio, including prestressing steel.

The volumetric ratio of transverse reinforcement given by Equation 3-34 may be used as a starting point, but the adequacy of the amount provided must always be checked by comparing displacement demand with capacity.

The pitch of spiral reinforcement provided for confinement should not exceed $6d_b$ nor $D_p/5$, where d_b is the diameter of the dowel reinforcement. For in-ground hinges in prestressed piles, the pitch should not exceed $3.5d_p$ where d_p is the nominal diameter of the prestressing strand.

Addition of Mild Steel Reinforcement to Prestressed Piles

The use of mild steel dowels to provide moment-resisting connections between piles and decks is common. It is also common to provide additional mild steel reinforcement throughout the length of the pile in the belief that this will enhance the performance of the in-ground hinge. Tests by Falconer and Park have shown this additional reinforcement to be unnecessary. Provided adequate confinement is provided at a pitch not greater than 3.5 times the prestress strand nominal diameter, dependable ductile response can be assured. Figure 3-29 compares lateral force-displacement hysteresis loops of prestressed piles with and without additional longitudinal mild steel reinforcement, and subjected to high axial load levels. Both piles were able to sustain displacement ductility levels of 10 without failure. The pile with additional mild steel reinforcement was, as expected stronger than the pile without additional reinforcement, and the loops indicated somewhat enhanced energy dissipation. However, in-ground hinges will normally only be subjected to moderate levels of ductility demand, for which the added damping provided by the mild steel reinforcement will be of only minor benefit.

It is thus recommended that additional longitudinal mild steel reinforcement be provided in piles only when there is a need to increase the flexural strength.

Capacity Protection of Elastic Actions and Members.

The essence of modern seismic design is the precise determination of where and how inelastic actions may occur (i.e. by inelastic flexural rotations in specified plastic hinge locations), and the protection of other locations (e.g. the deck) and other actions (e.g. shear) to ensure these remain elastic. This is termed Capacity Design, Priestley et al (1996) and is done by ensuring that the dependable strengths of the protected locations and actions exceeds the maximum feasible demand based on high estimates of the flexural strength of plastic hinges. Since development of flexural plastic hinge strength is certain at the design seismic input, the consequence of material strengths significantly exceeding design values will be that corresponding increases will develop in the forces of capacity protected members.

The most consistent method for determination of the required strength of capacity protected actions and members is to carry out a second series of pushover analyses, or dynamic time-history analyses, where the moment curvature characteristics of the pile plastic hinges are based on realistic upper bound estimate of material strengths. The following values are recommended:

Concrete compression strength	$f'_{cm} = 1.7\, f'_c$
Reinforcement yield strength	$f_{ym} = 1.3\, f_y$
Prestress strand ultimate strength	$f_{pum} = 1.1 f_{pu}$

The design required strength for the capacity protected members and actions should then be determined from the pushover analyses at displacements corresponding to the damage control limit state. Since these force levels will be higher than those corresponding to the serviceability limit state, there is no need to check capacity protection at the serviceability limit state.

A simpler, conservative approach to the use of a second "upper bound" pushover analyses is to multiply the force levels determined for capacity protected actions from the initial design analysis by a constant factor, representing the maximum feasible ratio of required strength based on upper bound and lower bound material strengths in plastic hinges. This ratio should be taken as 1.4, Priestley et al. (1996).

Shear Strength of Piles

The requirement for capacity protection is that the dependable strength exceeds the maximum feasible demand. Hence shear strength should be based on nominal material strengths, and shear strength reduction factors should be employed.

Most existing code equations for shear strength of compression members, including the ACI 318 equations which are widely used in the USA, tend to be unreasonably conservative, but do not adequately represent the influence of reduction of the strength of concrete shear resisting mechanisms. An alternative approach, Kowalski et al (1998), which has been widely calibrated against experimental data and shown to provide good agreement over a wide range of parameter variations is recommended for assessing the strength of piles. This approach is based on a three parameter model, with separate contributions to shear strength from concrete (V_c), transverse reinforcement (V_s), and axial load (V_p):

$$V_N = V_c + V_s + V_p \qquad (3\text{-}35)$$

A shear strength reduction factor of 0.85 should be applied to Equation 3-35 to determine the dependable shear strength.

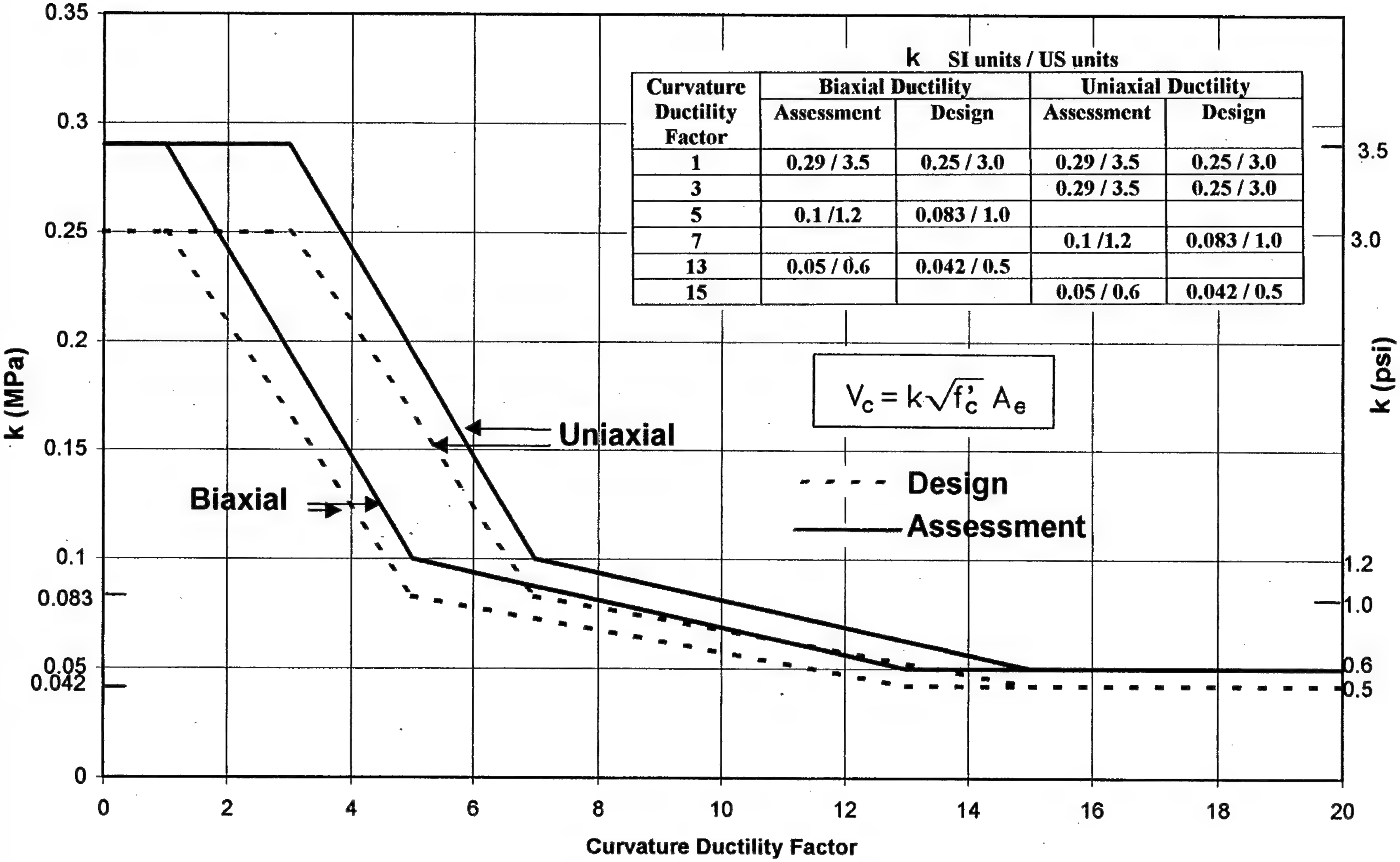
Concrete Mechanism Strength: The strength of the concrete shear resisting mechanisms, which include the effects of compression shear transfer, aggregate interlock, and dowel action, is given by:

$$V_c = k\sqrt{f'_c}.A_e \qquad (3\text{-}36)$$

Where k = factor dependent on the curvature ductility within the plastic hinge region, given by Figure 3-30,

$f'_c =$ concrete compression strength in MPa,

Figure 3-30. Concrete shear mechanism.



k SI units / US units

Curvature Ductility Factor	Biaxial Ductility		Uniaxial Ductility	
	Assessment	Design	Assessment	Design
1	0.29 / 3.5	0.25 / 3.0	0.29 / 3.5	0.25 / 3.0
3			0.29 / 3.5	0.25 / 3.0
5	0.1 /1.2	0.083 / 1.0		
7			0.1 /1.2	0.083 / 1.0
13	0.05 / 0.6	0.042 / 0.5		
15			0.05 / 0.6	0.042 / 0.5

A_e=0.8A_{gross} is the effective shear area.

The reduction in k with increasing curvature ductility $\mu_\phi = \phi / \phi_y$ occurs as a result of reduced aggregate interlock effectiveness as wide cracks develop in the plastic hinge region. For regions further than 2D_p from the plastic hinge location, the flexural cracks will be small, and the strength can be based on $\mu_\phi = 1.0$.

Different values of k are provided in Figure 3-30 for new design and for assessment. It is appropriate to be more conservative for new design than for assessment, since the economic consequences of extra conservatism are insignificant when new designs are considered, but can be substantial when assessment of existing structures are concerned. Different values are also given depending on whether the pile is likely to be subjected to inelastic action in two orthogonal directions (biaxial ductility) or just in one direction (uniaxial ductility).

Transverse Reinforcement (truss) Mechanism: The strength of the truss mechanism involving transverse spirals or hoops is given by:

Circular spirals or hoops:

$$V_s = \frac{\pi}{2} . A_{sp} f_{yh} (D_p - c - c_o) \cot \theta / s \qquad (3\text{-}37)$$

where A_{sp}= spiral or hoop cross section area
- f_{yh}= yield strength of transverse spiral or hoop reinforcement
- D_p= pile diameter or gross depth (in the case of a rectangular pile with Spiral confinement)
- c = depth from extreme compression fiber to neutral axis at flexural strength (see Figure 3-31)
- c_o= concrete cover to center of hoop or spiral (see Figure 3-31)
- θ = angle of critical crack to the pile axis (see Figure 3-31) taken as 30^0 for assessment, and 35^0 for new design.
- s = spacing of hoops or spiral along the pile axis.

Rectangular hoops or spirals:

$$V_s = A_h . f_{yh} (D_p - c - c_o) \cot \theta / s \qquad (3\text{-}38)$$

Where A_h= total area of transverse reinforcement, parallel to direction of applied shear cut by an inclined shear crack.

Axial Load Mechanism: The presence of axial compression enhances the shear strength by development of an internal compression strut in the pile between the compression zones of plastic hinges, whose horizontal component V_p opposes the applied shear force. Axial prestress also acts in similar fashion to enhance shear

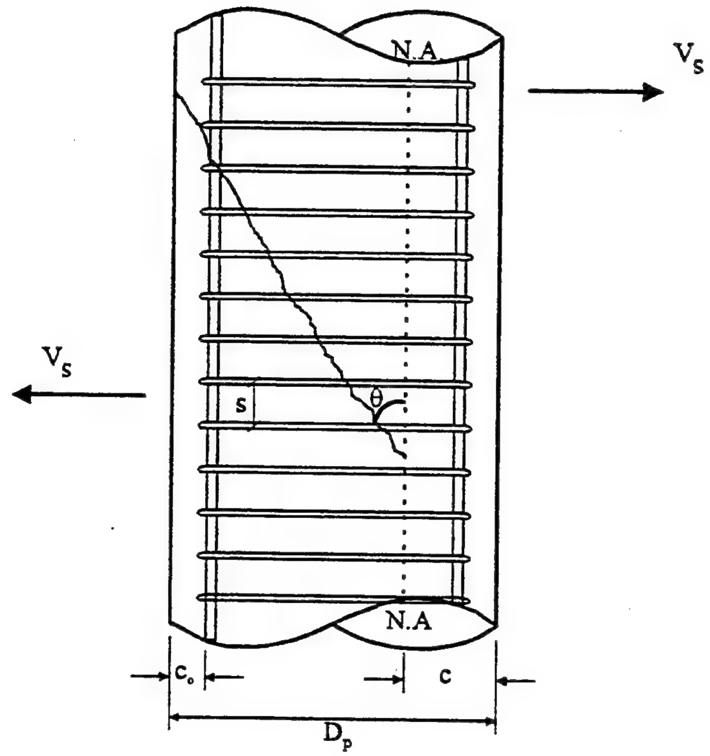


Figure 3-31. Transverse shear mechanism.

strength. Thus, with reference to Figure 3-32, the shear strength provided by axial compression is

$$V_p = \Phi\,(N_u+F_p)\tan\alpha \qquad (3\text{-}39)$$

Where N_u = external axial compression on pile including load due to earthquake Action,

F_p = prestress compression force in pile,

α = angle between line joining the centers of flexural compression in the deck/pile and in-ground hinges, and the pile axis (see Figure 3-32)

Φ = 1.0 for assessment, and 0.85 for new design.

Shear Strength of Concrete-filled Steel Shell Piles:

The flexural and shear strength of concrete-filled steel shell piles can be determined assuming normal reinforced concrete theory, and full composite action between the shell and the infill concrete. Although some slip between the shell and concrete may occur, it does not appear to significantly influence flexural strength or displacement capacity.

Shear strength can be determined using the equations above, and considering the steel shell as additional transverse hoop reinforcement, with area equal to the shell thickness, and spacing along the pile axis of s=1.0. The contribution of the shell to the shear strength is thus

$$V_{shell} = (\pi/2)\,t.f_{yh}(D_p - c - c_o)\cot\theta \qquad (3\text{-}40)$$

Design Strength for Deck Members

The required strength for deck members should be determined by adding the actions resulting from the "upper bound" pushover response, at the displacement levels corresponding to the level 2 earthquake, to those resulting from gravity action. Particularly in assessment of existing structures some redistribution of design actions is appropriate, provided overall equilibrium between internal actions and external forces is maintained.

Dependable strength for flexure and shear actions can be determined in accordance with ACI 318 principles.

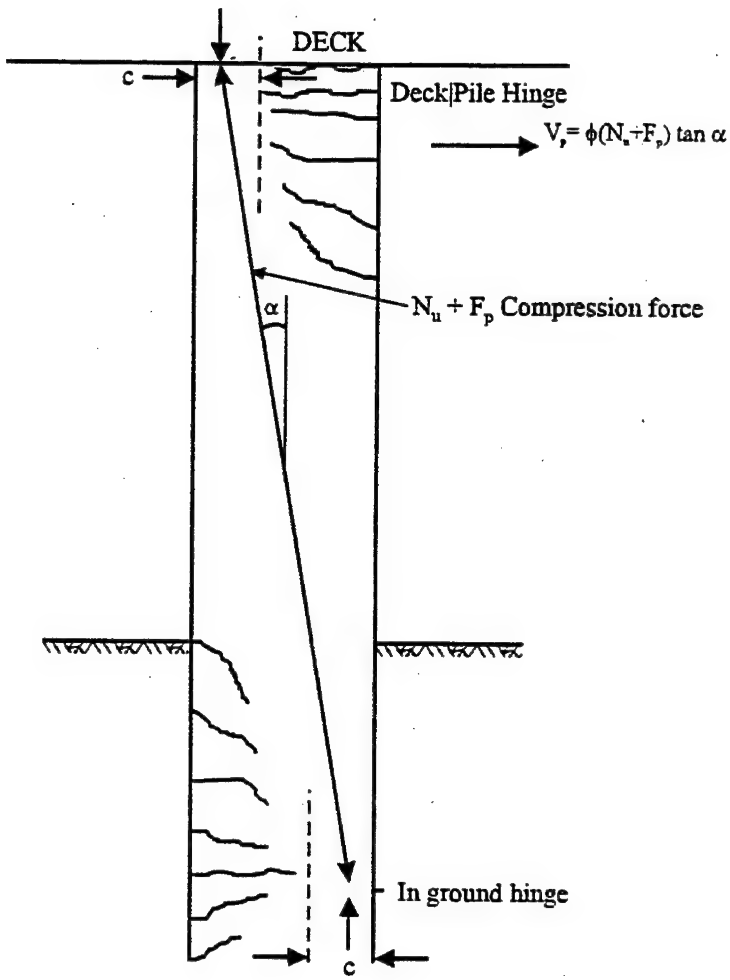


Figure 3-32. Axial force shear mechanism.

Assessment of Wharves and Piers with Batter Piles

Batter piles primarily respond to earthquakes by developing large axial compression or tension forces. Bending moments are generally of secondary importance. The strength in compression may be dictated by material compression failure, by buckling, or, more commonly by failure of the deck/pile connection, or by excessive local shear in deck members adjacent to the batter pile. Strength in tension may be dictated by connection strength or by pile pull out. In assessing the seismic performance of wharves and piers with batter piles the following additional items should be considered.

- Pile pull-out is ductile and has the potential to dissipate a considerable amount of energy. Displacement capacity is essentially unlimited.

- In compression, displacement capacity should consider the effect of reduction in pile modulus of elasticity at high axial load levels, and the increase in effective length for friction piles, resulting from local slip between pile and soil. Typical calculations indicate that displacement ductilities of 2 or more are possible.

- Where the prime concern is the prevention of oil spillage, t should be recognized that failure of the batter piles does not necessarily constitute failure of the wharf or pier, nor the initiation of oil spillage. It is possible that after failure of the batter piles, the wharf or pier may be capable of sustaining higher levels of seismic attack. Although the strength will be diminished as a consequence of the batter pile failure, the displacement capacity of the wharf or pier will generally be greatly increased before the secondary failure stage, involving the vertical piles, develops. Consequently this system, involving only the vertical piles should be checked independently of the batter pile system.

Assessment of Wharves and Piers with Timber Piles

There is little reported evidence of damage to timber piles in earthquakes, despite their wide spread usage for wharf and Pier supports for the past 150 years on the West Coast of the USA. This is due to their large displacement capacity, and the typically low mass of the supported wharf, if also constructed of timber, though there are cases of timber piles supporting concrete decks.

Extensive testing of timber piles, both new and used, in both dry and saturated conditions has been carried out by British Columbia Hydro (B.C.Hydro, 1992). No significant difference between results of new and old, wet and dry piles was found. The tests were carried out on nominal 12-inch (300mm) diameter. Peeled Douglas fir piles, though actual dimensions were as large as 14 inches (350mm) at the butt end, and as small as 9 inches (225mm) at the tip end. Three types of tests were carried out:

1. A simple bending test using a non-central lateral load on a pile simply supported over a length of 27.2 feet (8.28m).

2. Cantilever bending tests on piles embedded in concrete pile caps, with a free length of about 4 feet (1.2m)

3. In-situ testing of piles embedded in a firm silty-sand foundation, with a free length of about 16.4 feet (5m).

In all cases the piles were subjected to cyclic loading, though in only the first series of tests were the displacements equal in the opposite loading directions. All piles were subjected to axial loads of about 20 kips (89kn) throughout the testing.

The results indicated that the piles typically exhibited ductile behavior. Failure generally initiated by compression wrinkling at the critical section, followed by a period of essentially plastic deformation at approximately constant lateral load, terminated in a tension fracture. The first series of tests gave the lowest results in terms of displacement capacity. This may have been due to the steel loading collar, which applied lateral force to the pile at the location of maximum moment, causing local distress because of the sharp edges of the collar. In fact, failure was not achieved in the other two test series before actuator travel capacity was reached, despite displacement of 1.00m (40 in) for the in-situ pile tests.

Back-analyses from the more conservative, (and more extensive) first series (simply supported beams) indicated that the following data may conservatively be used for checking the capacity of existing Douglas fir piles:

Modulus of Elasticity:	10 GPa ($1.5x10^6$ psi)
Modulus of Rupture:	35 GPa (5000 psi)
Serviceabilty limit strain:	0.004
Ultimate limit strain:	0.008
Damping at ultimate limit:	12 %

The fracture strain has been back-calculated from the maximum displacements assuming a linear curvature distribution along the pile. This is because there is insufficient data to define a true ultimate strain, and an effective plastic hinge length. Thus displacement capacity should be based on the same assumptions. Note that this will result in the ultimate displacement capacity of a 12-inch (300 mm) pile with an effective length of 25 feet being about 40 inches (1000 mm).

Since the effectiveness of lateral bracing in timber wharves will generally be at best suspect, it is recommended that, as a first approximation, it conservatively be ignored when assessing seismic performance. This assumes that the bracing connections, or members, will fail or soften to the extent that they will be ineffective after the initial stages of lateral response. Similarly, it may be sufficient to assume that the connections between batter piles and the wharf fail, and to check the "worst case" condition without

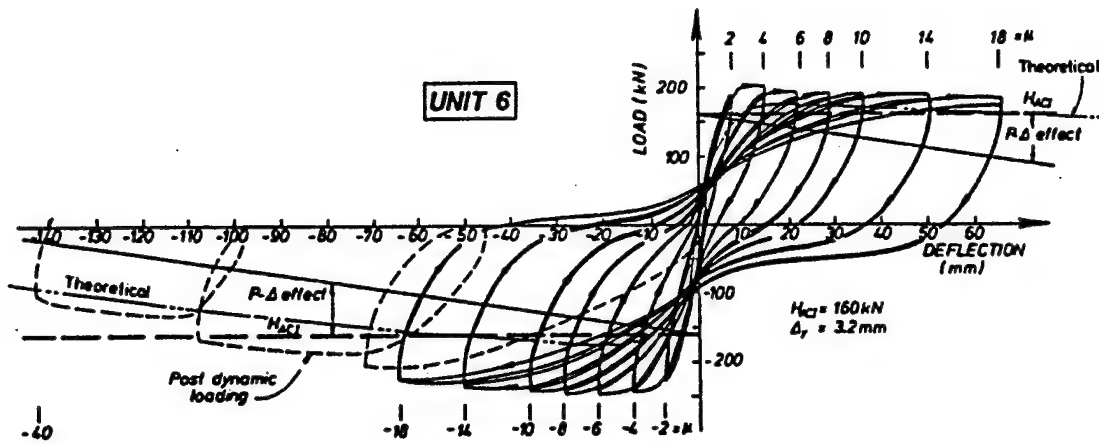


Figure 3-33. Force displacement response of concrete-filled steel shell pile connected to deck by dowels.

the batter piles. Generally, calculations based on these conservative assumptions will still produce displacement demands that are significantly less than displacement capacity.

Deck/Pile Connection Details

Connection details between the pile and the deck depend on the type of pile used to support the wharf or pier. Although connection details for buildings and bridges are often detailed to act as pinned connections, piles for wharves and piers are almost always designed for moment resistance at the pile/deck connection. Consequently only moment-resisting connections will be considered in this document.

Steel-Shell Piles

Steel shell piles will normally be connected to the deck via reinforcing bars and a concrete plug, even when the concrete infill is not continuous down the height of the pile. If the concrete plug is only placed in the vicinity of the connection, care is needed to ensure that shear transfer exists between the concrete and the steel shell. Although this may often be adequately provided by natural roughness of the inside surface of the steel shell, some more positive method of transfer should be considered. One possibility is the use of weld-metal laid on the inside surface of the steel shell in a continuous spiral in the connection region, prior to placing the concrete plug. Park et al (1983), investigating concrete filled steel-shell piles showed that dependable flexural strength and extremely large ductility capacity could be achieved when the steel shell was discontinuous 50mm inside the deck concrete, and the connection was made by dowel reinforcement properly anchored in the deck. An example of the force-displacement hysteresis response is shown in Figure 3-33. It will be observed that the flexural strength considerably exceeds the nominal strength, denoted H_{ACI} when P-Δ effects are included. This is partly a result of the enhanced concrete strength resulting from very effective concrete confinement by the steel shell, and partly due to the steel shell acting as compression reinforcement, by bearing against the deck concrete. The remarkable ductility capacity of the pile apparent in Figure 3-33 has been duplicated in other tests with discontinuous casings at the interface. This pile, subsequent to two cycles at a displacement ductility of $\mu_\Delta = 18$ was subjected to 81 dynamic cycles at $\mu_\Delta = 20$, and then a static half cycle to $\mu_\Delta = 40$ without significant strength degradation.

Prestressed Piles

In the USA, prestressed piles are normally connected to the deck with mild-steel dowel reinforcement, as discussed in the next section. However, other details are possible, and have been successfully tested under simulated seismic conditions.

Pam et al. (1988), tested a number of connection details appropriate for prestressed solid piles. The details were based on 400mm octagonal piles with 10-12.5mm tendons, but the results can be safely extrapolated to the 600mm piles more commonly used in wharf structures. Two units were tested with the solid end of the pile embedded 800mm into the deck, with a 10mm diameter spiral at 150mm pitch placed around the embedded length of the pile. Two further piles were tested with the strand exposed, enclosed in a 12mm spiral at 47mm pitch, and embedded straight in the deck for a distance of 600mm. In each of the pairs of piles one pile contained only strand while the other included 10-20mm diameter mild steel dowels, thus increasing the flexural strength of the piles. All pile units were subjected to an axial load ratio of $N_u/f'_c A_{gross}$ = 0.2, in addition to the axial compression resulting from prestress.

Results for the force-displacement response of these four units are shown in Figure 3-34. In each case the piles were capable of developing the theoretical moment capacity of the connection, and to maintain it to high ductility levels. The apparent strength degradation in Figure 3-34 is a result of P- Δ effects from the moderately high axial load.

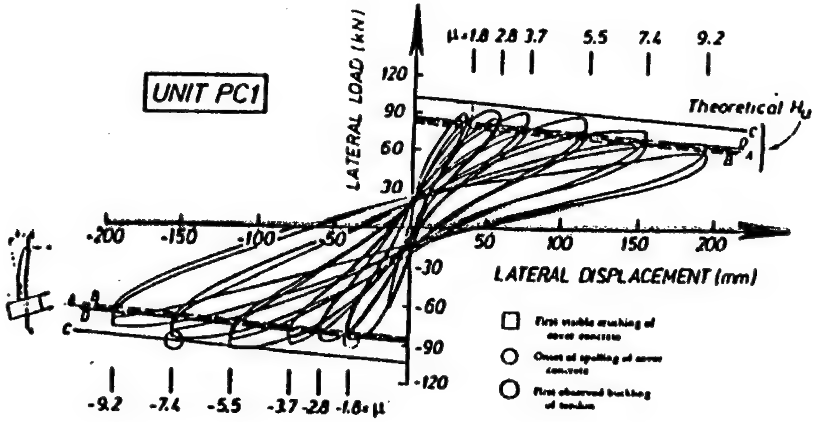
First sign of crushing of the concrete in the plastic hinge region was noted at displacement ductility factors of about 2.0 for the embedded pile and about 2.5 for the embedded strand. The difference was due to the higher total compression force at the critical section (due to the pile prestress force) when the full pile section was embedded in the pile cap. Since the piles were very slender, with an aspect ratio of 7, these values might be considered as lower bound estimates for a serviceability criterion for the piles. However, this would be compensated by increased yield displacements if the flexibility of the deck had been included in the tests.

In all cases, the strand was fully developed, despite the rather short development length of $48d_b$ in the exposed strand tests. These results indicate that strand is sufficient to develop moment capacity at the deck/pile interface, and that mild steel dowels are not necessary.

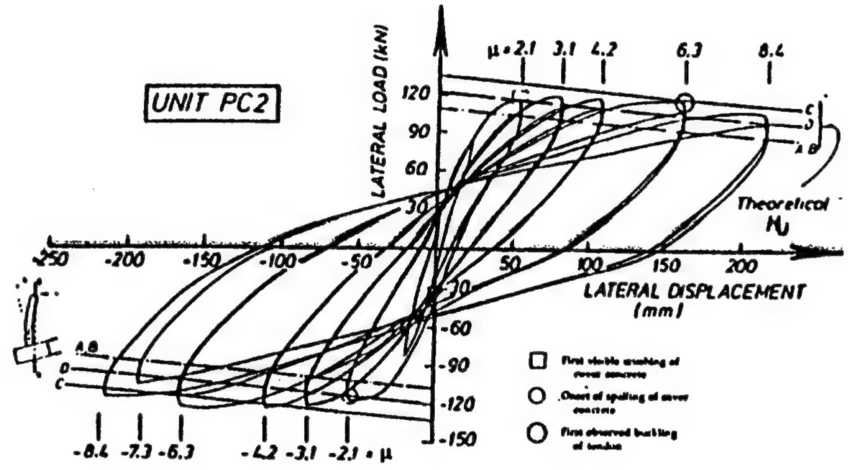
The test units by Pam et al had rather light joint reinforcement surrounding the pile reinforcement in the joint region. However, it must be realized that the deck in these tests was rigidly connected to a reaction frame, which reduced stress levels in the joint region. Practical details should thus not be based on the joint rebar in Pam's tests.

Practical Connection Considerations.

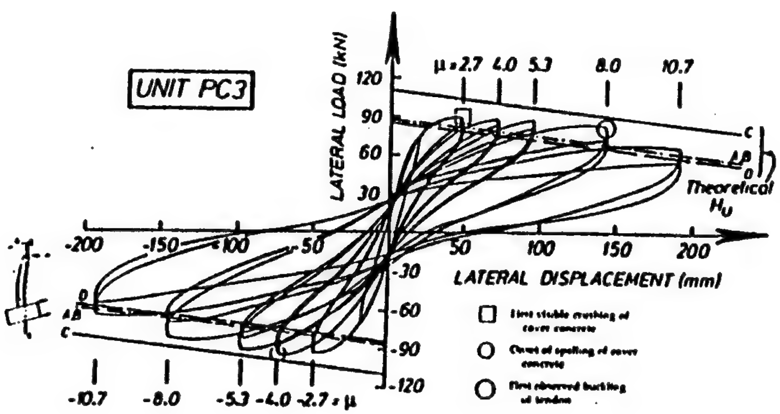
The most common connection detail consists of dowel reinforcing bars, typically of size #8, #9, or #10, inserted in ducts in the top of the pile after the pile has been driven to the required level, and the top cut to the correct final elevation. Typically the dowel bars in the past have been bent outwards, with the top of the bend below the level



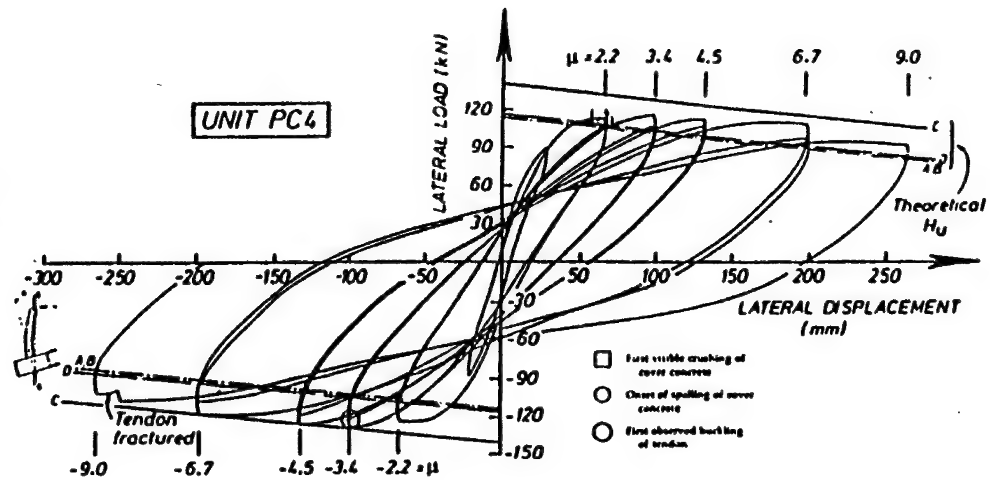
a) Solid pile embedded 800 mm, no dowels



b) Solid pile with standard dowels embedded 800 mm



c) Exposed strand embedded 600 mm, no dowels



d) Exposed strand and dowels embedded 600 mm

Figure 3-34. Lateral force-displacement response for four pile-pile cap connection details (D=400mm) tested by Pam et al.

of the top mat of the deck reinforcement, as suggested in Figure 3-35(a). In the past the top of the bend has often been much lower than shown in this figure.

If hooks are provided on dowels, they must have the tails bent inwards, rather than outwards. The reason for this is that if significant tension force is transferred up to the hook, which is bent outwards, it adds tension stress within the joint region which is already subjected to high tension stress as a result of joint shear forces. There is then a tendency for the diagonal joint shear crack to propagate and bend horizontally outside the hook, particularly if the hooks are below the top layer of deck reinforcement, as illustrated in Figure 3-35(a). The problem is compounded if the top of the dowel hooks is lower in the deck than shown in Figure 3-35(a). Proper force transfer between the pile and deck becomes increasingly doubtful, and spalling of the soffit concrete may occur, as has been observed in response to moderate earthquakes in Southern California.

If the hooks are high, and are bent inwards as shown in Figure 3-35(b), the bend results in anchorage forces directed back towards the compression corner of the deck/pile connection, resulting in a stable force-transfer mechanism.

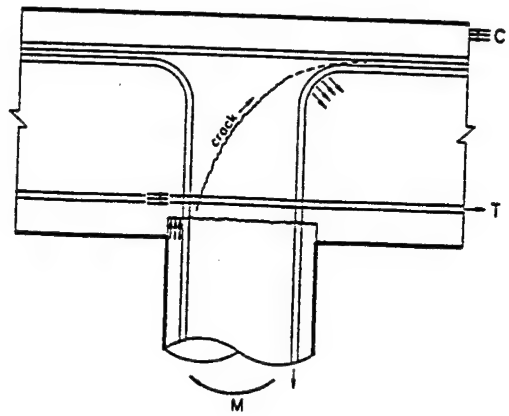
Although satisfactory force transfer is expected with the detail of Figure 3-35(b), it can be difficult to construct. This is because tolerances in the final position in plan of a driven pile may be as high as +/- 150mm. In such cases interference between the bent dowels and the deck reinforcement can cause excessive placing difficulties, which may result in one or dowel bars being omitted. Design details need to be developed that are simple and insensitive to pile position. This means that bent dowels should not generally be used.

Two alternative details, developed for the Port of Los Angeles, Priestley (1998) are shown for typical 600mm prestressed piles in Figure 3-36. The first uses straight bar development up as high as possible into the deck, but with bars not terminating more than 100mm below the top surface. This is combined with spiral reinforcement to control the potential joint shear cracking. The second uses reduced embedment length of the dowels, with partial anchorage provided by enlarged end upstands (end bulbs), with bars lapped with headed vertical bond bars. The lap and joint are again confined by a spiral.

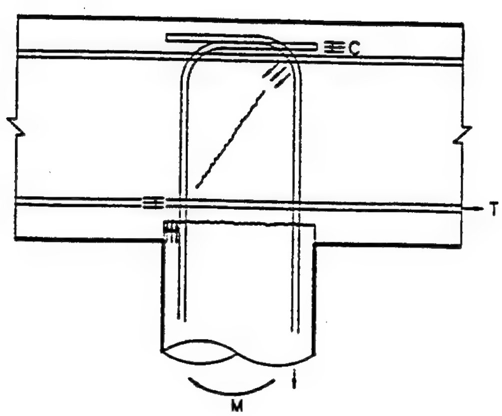
Straight-bar Embedment Detail: The straight bar embedment detail of Figure 3-36(a) is directly analogous to details used for moment-resisting column/cap-beam connections for seismic response of bridges, Priestley et al. (1996). The required embedment length is thus given by

SI units $$l_e = 0.3 d_b \frac{f_{ye}}{\sqrt{f'_c}}$$ (Mpa, mm) (3-41a)

US units $$l_e = 0.025 d_b \frac{f_{ye}}{\sqrt{f'_c}}$$ (psi, in.) (3-41b)

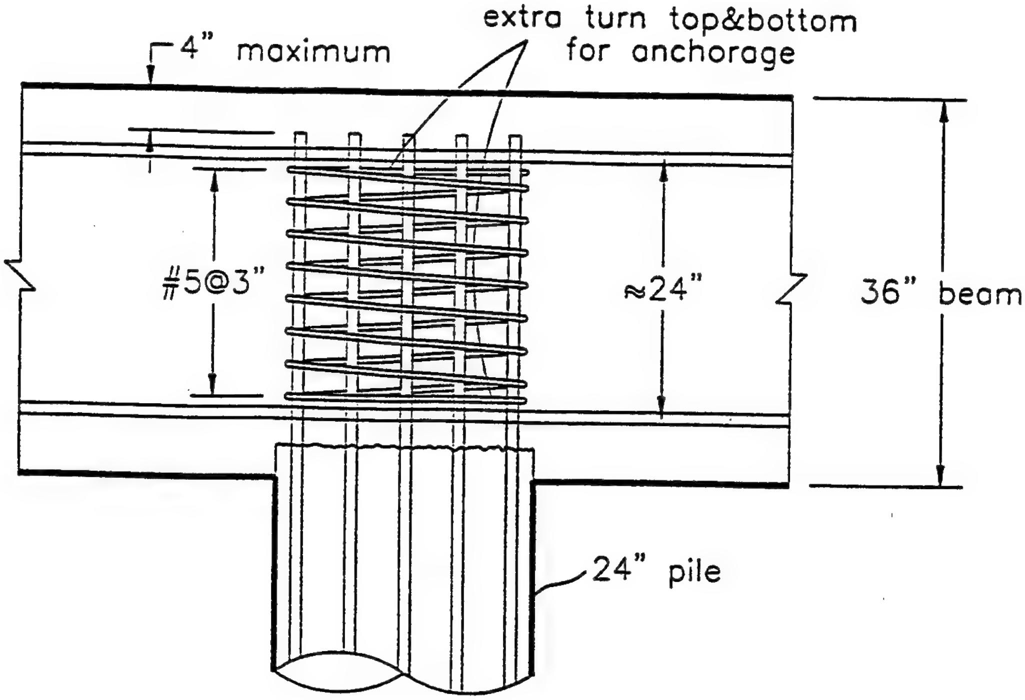


(a) hooks, low, bent out

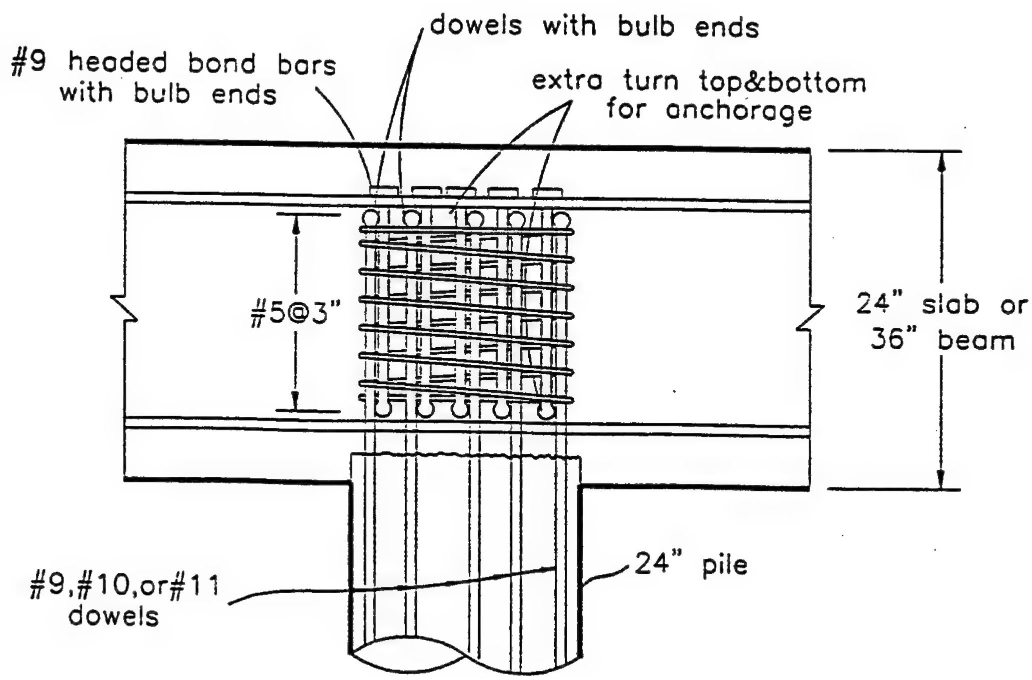


(b) hooks high, bent in

Figure 3-35. Anchorage with hooked dowels.



(a) Anchorage detail for dowels in 36-inch beam



(b) Anchorage detail for dowels with upstands.

(Note: Take dowels up to as high as possible: 3 in. from C.G. top or bottom mat okay)
#9 headed bond bars: 30 in. long for 36 in. beam 18 in. long for 24 in. beam

Figure 3-36. Anchorage details for dowels.

where the dowel bar diameter l_e is in mm, and the material strengths are in MPa.

For typical concrete strengths of about f'_c = 30MPa and dowel strength of f_{ye} = 450MPa, Equation 3-41 indicates that it is feasible to anchor #8 to #10 dowels by straight embedment in a 900mm deep deck beam, allowing 100mm from the top of the dowel to the deck surface. Anchorage of a #11 bar in a 900mm deck would require a somewhat higher deck compression strength. Note that the use of the specified 28 day concrete compression strength in Equation 3-41 is very conservative, given expected conservatism in concrete batch design, and expected strength gain before occurrence of the design earthquake, and it would be more realistic, particularly for assessment of existing structures to use a more characteristic strength, say $1.2f'_c$. This would still be considerably less than the probable strength at the age of the concrete when subjected to seismic loading.

Priestley (1998) requires that, if additional external joint reinforcement is not provided, the anchored bars must be enclosed in spiral or hoop confinement in accordance with

$$\rho_s = \frac{0.46A_{sc}}{D'l_a}\left[\frac{f^o_{yc}}{f_s}\right] \tag{3-42}$$

where f^o_{yc} =1.4f_y is the dowel overstrength bar capacity,
f_s = 0.0015E_s,
l_a =actual embedment length provided,
A_{sc}= total area of dowel bars in the connection
E_s = dowel modulus of elasticity
D'= diameter of the connection core, measured to the centerline of the spiral confinement.

The detail shown in Figure 3-36(a) is adequate for embedment of 8 #11 dowels in a 900mm deck beam.

Headed Rebar Detail: This detail, shown in Figure 3-36(b), is designed to allow the dowels to terminate below the top layer of beam reinforcement. Anchorage is improved by using an upstand on the end of the dowel. In this design, 50% of the dowel anchorage force is transferred by the upstand, and 50% by bond, using special headed reinforcement bond bars. This allows the minimum embedment length to be reduced to 50% of that given by Equation 3-41. The top of the dowel should be as high as practicable, but in this case may be below the top mat of reinforcement. With this detail, all bar sized up to #11 can safely be anchored in a 900mm deck beam, and all sizes up to #10 can be anchored in a 600mm deep deck beam.

The total required area of the headed bond bars is $A_t = 0.65A_{sc}$. Spiral confinement around the connection must be provided in accordance with Equation 3-42.

Details similar to those shown in Figure 3-36(a) have been tested for bridge designs, Sritharan and Priestley (1998) and found to perform well, with displacement ductilities typically exceeding $\mu_\Delta = 6$. The detail of Figure 3-36(b) was recently tested for the Port of Los Angeles, and found to have excellent response (see Figure 3-37), with failure finally occurring outside the connection at high displacement ductility. Only minor cracking developed in the joint region.

Steel H-Section Piles. Connection of steel H-Piles to pile caps or decks is normally by partial embedment in the deck. Full moment-resisting connections are rarely attempted with this detail. If moment-resistance is required of the connection, sufficient transverse reinforcement must be placed around the pile head to enable the pile moment to be transferred by bearing in opposite directions at the upper and lower regions of the pile embedment.

Capacity of Existing Substandard Connection Details

Many existing piers and wharves will have connection details similar to that shown in Figure 3-35a, where the dowels bend outwards from the pile centerline, and the top of the 90 degree bend, h_d above the soffit, is well below the deck surface. Typically, the dowels will not be enclosed in a spiral within the joint region, and there is thus a probability of joint shear failure.

There are no known test data related to the detail of Figure 3-35, but poor performance of such details in the recent moderate Northridge earthquake, and similarity to bridge T joints, which have been extensively tested, leads to a need for conservative assessment.

A relevant procedure is available in Priestley et al. (1996), and is adapted in the following for wharves and piers with reinforced concrete decks.

1. Determine the nominal shear stress in the joint region corresponding to the pile plastic moment capacity.

$$v_j = \frac{M^o}{\sqrt{2}\ h_d D_p^2} \tag{3-43}$$

where M_o is determined as described in "Capacity Protection of Elastic Actions and Members" above and h_d is defined above and in Figure 3-35a.

2. Determine the nominal principal tension stress in the joint region:

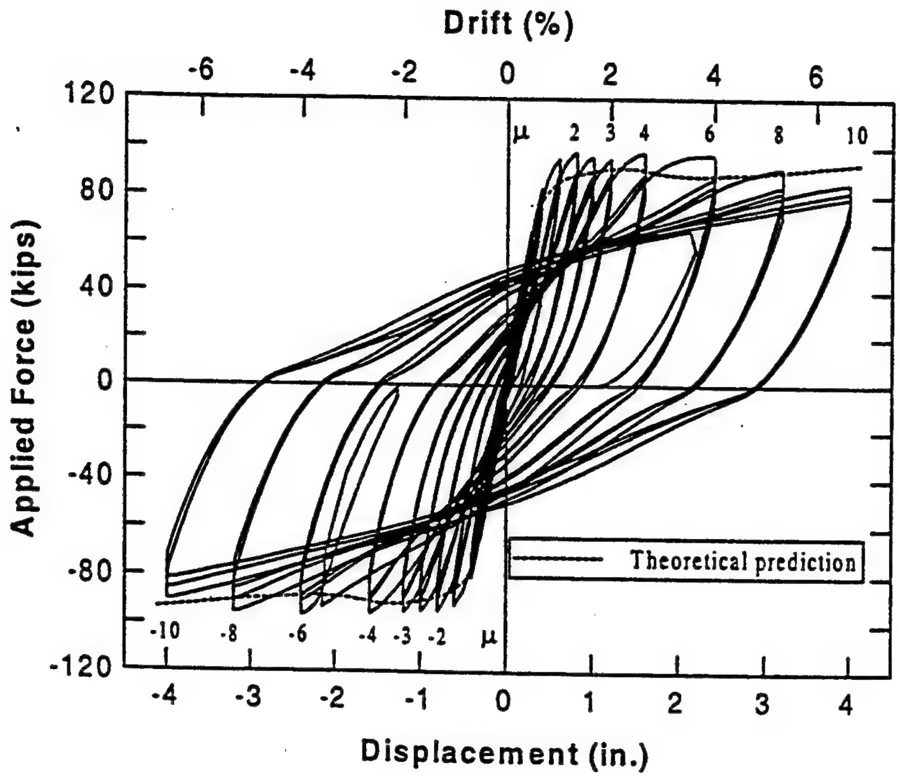


Figure 3-37. Force – displacement response of pile connected to deck with headed rebar (detail of Figure 3-36b.)

$$p_t = \frac{-f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_j^2}$$

(3-44)

where

$$f_a = \frac{P}{(D_p + h_d)^2}$$

(3-45)

is the average compression stress at the joint center, caused by the pile axial compression force P. Note, if the pile is subjected to axial tension under the seismic load case considered, then the value of P, and f_a will be negative.

3. If p_t calculated from Equation 3-44 exceeds 0.42 $\sqrt{f_c'}$ Mpa, joint failure will occur at a lower moment than the column plastic moment capacity M^o. in this case, the maximum moment that can be developed at the pile/deck interface will be limited by the joint principal tension stress capacity, which will continue to degrade as the joint rotation increases, in accordance with Figure 3-38. The moment capacity of the connection at a given joint rotation can be found from the following steps.

4. From Figure 3-38, determine the principal tension ratio $p_t / \sqrt{f_c'}$ corresponding to the given joint rotation, referring to the T-joint curve.

5. Determine the corresponding joint shear stress force from:

$$v_j = \sqrt{p_t(p_t - f_a)}$$

(3-46)

where f_a is determined from equation 3-45.

6. The moment at the pile/deck interface can be approximated by:

$$M_c = \sqrt{2}\; v_j h_d D_p^2 \le M^o$$

(3-47)

This will result in a reduced strength and effective stiffness for the pile in a push-over analysis. The maximum displacement capacity of the pile should be based on a drift angle of 0.04.

Discussion Of Criteria

The criteria presented above are developed from a compilation of current practice by many agencies combined with state-of-the-art technology for estimation of seismic

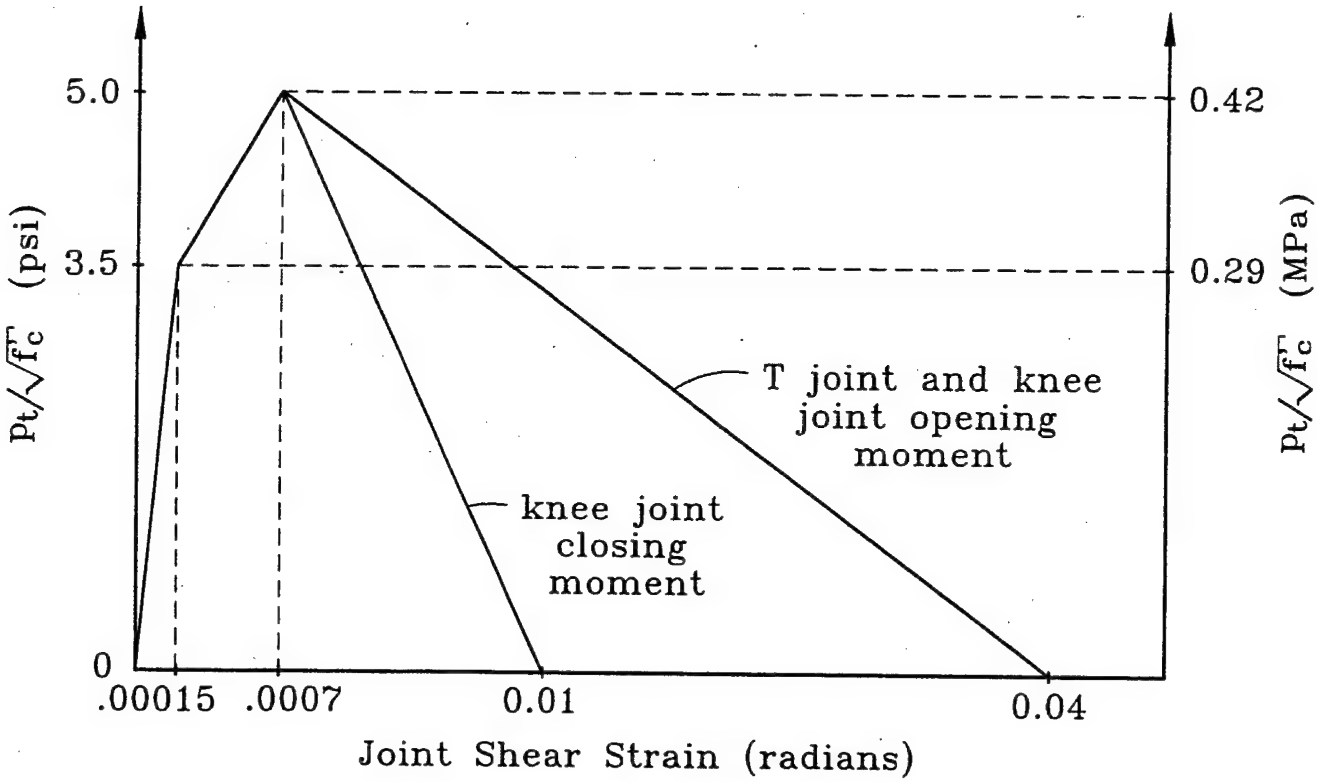


Figure 3-38. Degradation of effective principal tension strength with joint shear strain. (After Priestley, Seible, and Calvi, 1996)

damage potential. It is not a revolutionary step forward but rather an evolution of design. This specification has developed a cohesive integrated criteria specifying:

1. The required pier performance under expected loads
2. Specification of the expected loads
3. Specification of strain limits to ensure structural response limits to achieve performance requirements.

The overall effect on the design, selection of pile sizes and cost of the pier is not expected to be great; however, the assurance in meeting performance goals is thought to be substantially enhanced.

In the application of these criteria to existing construction, it is thought that the objective of a uniform set of performance goals should be maintained across the waterfront. Where adequate capacity is lacking in the existing system, it is thought better to strive for the performance goal, develop several candidate upgrade alternatives, and then perform an economic/risk analysis to determine what is the most cost effective solution considering the potential for a damaging earthquake and the existing lateral force system. This approach is preferred over any system which arbitrarily establishes some percentage reduction of a new-construction criteria. Any single reduction coefficient is probably not optimal over a range of structures and is at best arbitrary.

Strengthening of an Existing Structure

Various methods of strengthening existing structures are possible:

Plating. Where the top of flange is not accessible for adding cover plates, reinforcement can be added to the web plate. The beam shall be relieved of load before the reinforcement is added. When cover plates are added, the flange to web connection and the web plate stresses at the toe of the flange shall be investigated.

Composite Action. Beam section properties can be materially increased by causing the concrete slab to act as a composite with the beam. The slab serves as a top cover plate.

Prestressing Jacks can be used effectively to reduce stresses in existing flanges. Cover plates are welded before removing jacks.

Shear Reinforcement. Vertical stirrups serve as hangers that support the beam from the uncracked portion of concrete near the column.

Flexural Reinforcement. Longitudinal reinforcement can be added effectively if positive means for preventing separation and for transferring horizontal shear are used.

Composite materials can be glued to the underside of pier decks to increase section capacity. Composite rods can be inserted and epoxied in groves cut in the topside of pier decks to add reinforcement.

Pile/Column Reinforcement. Pile/Column sections may be strengthened by adding concrete with longitudinal and lateral reinforcement or by adding unreinforced concrete restrained by hoop bars. Wrapping by composite materials has been used very effectively for bridge columns. Mandrels have been applied to piles and filled with concrete.

Compatibility The design details shall encompass any inherent incompatibility of old and new materials. Provision shall be made to resist separation forces. flew concrete shall have a different modulus of elasticity, coefficient of thermal expansion, and shrinkage than old concrete. Consider differing expansion effects due to differing absorption of moisture. Provide resistance against "curling" due to thermal gradients.

Compatibility of connectors must be considered. For example, rivets or bolts are not compatible with welds. Friction bolts are not compatible with rivets. Creep is an important factor.

Dead Load Versus Live Load Stresses. Unless the load on a structure is relieved (for example, by removal or by jacking), the existing framing will continue to carry:

a) the full dead load of the construction,

b) any part of the live load which is in place when the new framing is connected, and

c) a proportionate share of the live load subsequently added.

The new framing will carry only a part of the live load. As a result, under the final loading condition, the stresses in the new and existing material of the same or similar members will be different, often radically so. For example, assuming a 1:1 ratio of dead to live load and of new to existing material in the cross section of a given member and disregarding plastic deformation, the stress in the existing material would be three times the stress in the new. As a result, the new material cannot be stressed up to allowable values without simultaneously overstressing the existing sections. It is necessary either to provide an excess of new material or to relieve the load on the structure before strengthening. This may not apply if plastic deformation of the structure (and its associated, increased deflection) can be permitted.

Deterioration Of Waterfront Structures

The more common causes of deterioration associated with steel, concrete, and timber waterfront structures are as follows.

Steel Structures. For steel structures deterioration is caused by:

a) corrosion
b) abrasion
c) impact.

Concrete Structures. In concrete structures deterioration is caused

a) corrosion of reinforcement,
b) chemical reactions
c) weathering
d) swelling of concrete, and
e) impact

Timber Structures. In timber structures deterioration is caused by:

a) corrosion and abrasion of hardware
b) borer attack,
c) decay, and
d) impact.

Preventive Measures in Design and Construction

Steel Structures All parts that will be subject to corrosion should be accessible for inspection and repair. If not accessible, encase with concrete or provide some other long-life, high-resistance type of coating. Shapes shall be selected that have a minimum of exposed surface. Detailing shall be designed so that accumulations of dirt and debris will be avoided. Avoid narrow crevices that cannot be painted or sealed. Draw faying surfaces into tight contact by use of closely spaced stitch rivets, bolts, or welds. Prime faying surfaces before assembly. In general, detailing framing to shed water is the single most important factor in inhibiting corrosion and deterioration of coatings. If the potential for ponding is unavoidable, provide drain holes. Drain holes shall be a minimum of 4-inch (101.6 mm) in diameter to inhibit clogging. The use of sacrificial metal shall be avoided in favor of using protective coatings.

The thickness of metal and section properties shall be determined from consideration of loss of section as established in MIL-HDBK-1003/3, Steel Structures, unless corrosion protection is provided. Typical average corrosion rates for bare carbon steel are as follows:

Zone	Average Corrosion Rate (mills per year)
Imbedded Zone 0 to 15 ft	2
Erosion Zone 15 to 21 ft	10
Immersed Zone 21 to 29 ft	8
Atmospheric Zone 29 to 35 ft	12

The minimum thickness shall not be less than 0.40 inches (10.55mm). When the required minimum thickness is excessive, corrosion protection using approved products or cathodic protection shall be used. When coatings are used care must be exercised in driving the piles to preclude damage to the coatings. Additionally consideration must be given to abrasion of the piles by contact with fendering. Tips of all steel H piles having a thickness of metal less than 0.5 inches (12.7mm) and driven to end-bearing on sound rock by an impact hammer shall be reinforced.

Concrete Structures Good quality is the important factor in obtaining a dense concrete. This, in turn, is the most important factor in preventing penetration of moisture, which is the primary cause of deterioration of concrete. Do not use poorly graded aggregate, or a water-cement ratio greater than 6 gal (22.71 1)/sack of cement, reduced to 5 gal (18.92 1)/sack of cement for thinner sections such as slabs and wherever clear cover over reinforcement is 2 in. (50.8 mm) or less. Watertight concrete can be obtained by using air entrainment (maximum 6 percent by volume) and a water-cement ratio not greater than 5 gal/sack of cement. Type III (high early strength) cement is excessively susceptible to sulfate attack, and shall not be used. In general, avoid the use of Type I cement in a saltwater environment. Type IX (sulfate-resistant) cement shall be used. The use of Type V (high sulfate-resistant) cement is seldom required. Provision shall be made for an adequate number of expansion joints. Use types of expansion joints such as double bents with movement taken up by bending of the piles or elastomeric pads with some form of joint sealer. In tropical climates and in areas subject to salt spray, consider the use of galvanized or plastic coated reinforcing bars. If plastic coated bars are used, attention should be given to bond stresses. Excessively rich mixes, over 6 bags per yd^3 (0.764 m^3), shall be avoided, as excess cement tends to enhance the potential chemical reaction with seawater. For most aggregates, alkali-aggregate reaction can be prevented by specifying maximum alkali content of the cement (percent Na_20, plus 0.658 times percent K_20) not to exceed 0.60 percent. In a surf zone, the concrete cover and streamline sections shall be increased to prevent abrasion. Calcium chloride (as an accelerator) shall not be used in prestressed concrete and concrete exposed to seawater. The use of calcium nitrite or other chemicals as a deterrent to corrosion of embedded reinforcing steel is not an adequate substitute for good quality concrete and adequate cover. This is not to say

that these additives do not have merit; however, use of coated reinforcing bars may be required. Timber jackets for concrete piles and stone facing for concrete seawalls work extremely well to prevent deterioration due to corrosion of reinforcement, weathering, and chemical attack. They tend to isolate the concrete from chemical constituents in the environment, insulate against freezing, and keep free oxygen from the reinforcing bars.

Reinforced concrete has been used as one of the major construction materials at the waterfront. Since concrete is much weaker in tension, cracking would be expected to occur when the tensile stresses in the concrete were exceeded, typically at numerical level equal to about 10 percent of the maximum compressive stress. Cracking is a normal occurrence in concrete members under flexural load. When the concrete cracks the section moment of inertia is reduced; generally the cracked moment of inertia is about 30 to 60 percent of the gross moment of inertia depending on the axial load level and reinforcement content. In a marine environment it is desirous to control the cracking to prevent corrosion of the reinforcing steel. Confining steel is used to increase concrete strength, ductility and shear strength. An initial prestress force is used in piles as a mechanism for improving concrete performance by keeping the cracks closed. It has been noted that crack widths of 0.007 to 0.009 inches are sufficiently small to preclude deterioration of the reinforcement so an allowable crack width may be approximated at about 0.01 inches. It is not possible to directly equate the crack width to an allowable tensile strain since crack spacing is not known; however, corrosion has not been a problem when reinforcing stress has been restricted to a tension of 17 ksi or less under service loads. At concrete compressive strains below 0.0021 in/in the compression concrete does not evidence damage and crack widths under cyclic load should be acceptable. Occasional larger loads may be sustained without deterioration as long as a permanent offset does not occur and the prestress forces can close the cracks. Reinforcement deterioration is most pronounced in the presence of oxygen such as in a pier pile where the pile is freestanding out of water or in the splash zone. At deep-water depths or in soil, the oxygen content is reduced such that pile reinforcement deterioration is less. Large loads causing loss of the concrete cover result in loss of pile capacity and facilitate deterioration; such conditions can be repaired if accessible by jackets around the pile. Loss of concrete cover may begin at displacement ductilities of about 1.5 to 2.0.

Timber Structures. Timber structures shall conform to the following criteria:

a) Design detail shall minimize cutting, especially that which must be done after treatment.

b) Design detail shall provide for ventilation around timbers. Avoid multiple layers of timbers as decay is enhanced by moist conditions at facing surfaces. Curb logs shall be set up on blocks. Walers shall be blocked out from face of pier. Thin spacers between chocks and wales, and gaps between deck and tread planks shall be provided.

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CHAPTER 4 EVALUATION OF HAZARDS ASSOCIATED WITH SOIL LIQUEFACTION AND GROUND FAILURES

Soil Liquefaction Hazards

Introduction

Experience from past earthquakes demonstrates the vulnerability of waterfront structures and lifelines to seismically induced ground deformations. In an extensive review of the seismic performance of ports, Werner and Hung (1982) concluded that by far the most significant source of earthquake damage to waterfront structures has been pore pressure build-up in loose to medium-dense, saturated cohesionless soils that prevail in coastal and river environments. This observation has been supported by the occurrence of liquefaction-induced damage at numerous ports in the past decade (e.g., Seed et al. 1990; Chung, 1995; Mejia et al., 1995; Werner and Dickenson, 1996). Components of marine facilities conspicuous for poor performance due to earthquake-induced liquefaction include pile supported structures, sheet-pile retaining walls and bulkheads, and gravity retaining walls founded on or backfilled with loose sandy soils. The observed damage patterns commonly reflect the deleterious effects of (a) poor foundation soils which may have marginal static stability and which also tend to amplify the strong ground motions at these sites, and (b) the combination of high ground water levels and the existence of very loose to medium dense, sandy backfill and foundation soils. Saturated, sandy soils are susceptible to earthquake-induced *liquefaction*, a state wherein excess pore water pressures generated in the soil result in a temporary reduction, or complete loss, of strength and stiffness of the soil.

The liquefaction of a loose, saturated granular soil occurs when the cyclic shear stresses/strains passing through the soil deposit induce a progressive increase in the pore water pressure in excess of hydrostatic. During an earthquake the cyclic shear waves that propagate upward from the underlying bedrock induce the tendency for the loose sand layer to decrease in volume. If undrained conditions during the seismic disturbance are assumed, an increase in pore water pressure and resulting decrease of equal magnitude in the effective confining stress is required to keep the loose sand at constant volume. The degree of excess pore water pressure generation is largely a function of the initial density of the sand layer and the magnitude and duration of seismic shaking. In loose to medium dense sands pore pressures can be generated which are equal in magnitude to the confining stress. At this state, no effective (or intergranular) stress exists between the sand grains, and a complete loss of shear strength is temporarily experienced. The potential for the development of large strain (or flow) behavior is controlled by the initial relative density of the soil. The phenomena associated with the loss of strength of the sandy soils (e.g.; loss of bearing capacity, lateral spread, increase in active lateral earth pressures against retaining walls, loss of passive soil resistance below the dredge line and/or adjacent to anchor systems, and excessive settlements and lateral soil movements) contribute to the excessive deformations of waterfront structures. The large deformations associated with the failure of waterfront retaining walls can result in damage to wharf and backland, adversely affecting the operation of marine oil terminals.

One pertinent example, Figure 4-1, is provided by the observations made at the U.S. Naval Station at Treasure Island after the 1989 Loma Prieta earthquake. Inspection of the acceleration record obtained at the Treasure Island Fire Station shows that at about 15 seconds after the start of recording, the ground motion was subdued; this was probably caused by the occurrence of subsurface liquefaction. Liquefaction occurred after about 4 or 5 "cycles" of shaking, about 5 seconds of strong motion. Sand boils were observed at numerous location and bayward lateral spreading occurred with associated settlements. Ground cracking was visible with individual cracks as wide as 6 inches. Overall lateral spreading of 1 foot was estimated. Ground survey measurements indicate that settlements of 2 to 6 inches occurred variably across the island and that some areas had as much as 10 to 12 inches of settlement. The liquefaction related deformations resulted in damage to several structures and numerous broken underground utility lines, Egan (1991).

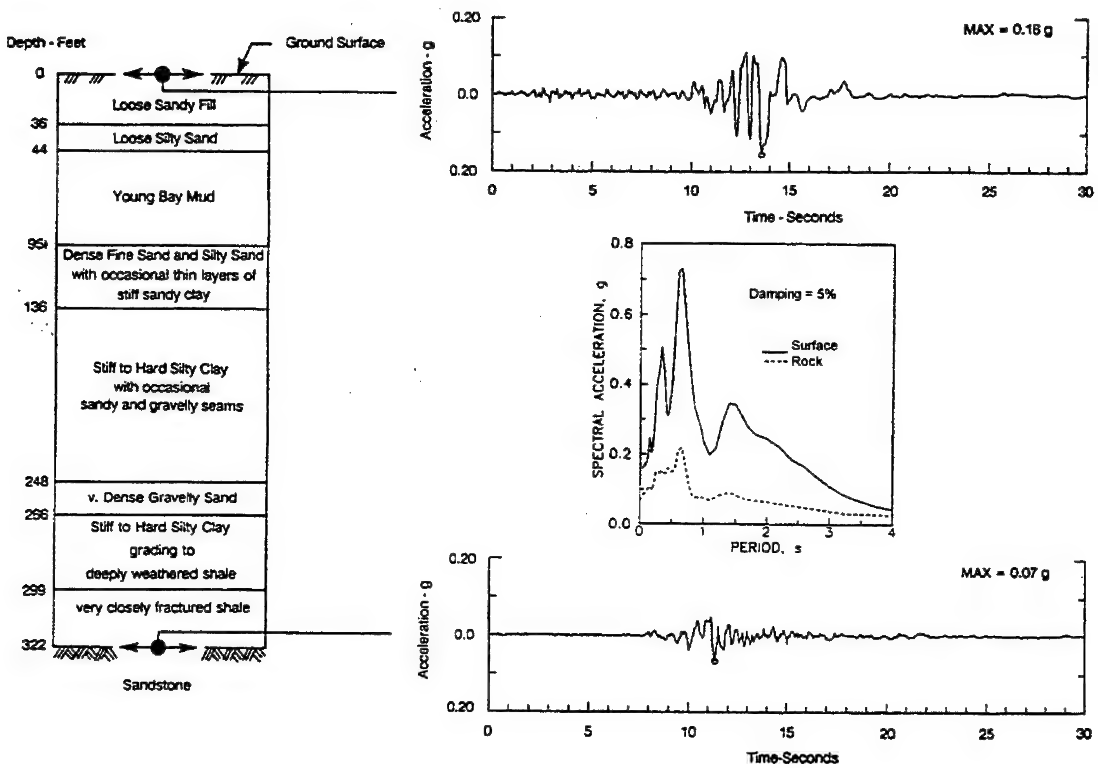

Figure 4-1: Soil Response at Treasure Island During the 1989 Loma Prieta Earthquake (After Seed et al., 1990)

Evaluation of Liquefaction Susceptibility

Experience from liquefaction-induced damage to structures and lifelines during past earthquakes shows that liquefaction hazards can be broadly classified into three general modes: (a) global instability and lateral spreading; (b) localized liquefaction hazard; and (c) failure or

excessive deformation of walls and retaining structures. These liquefaction hazards may vary dramatically in scale, and the extent of each of these potential soil failure modes must be evaluated for projects in seismically active regions. The scope of the investigation required will reflect the nature and complexity of the geologic site conditions, the economics of the project, and the level of risk acceptable for the proposed structure or existing facility.

The evaluation of liquefaction hazard is generally performed in several stages that include: (a) preliminary geological/geotechnical site evaluation; (b) quantitative evaluation of liquefaction potential and its potential consequences; and, if necessary (c) development of mitigation and foundation remediation programs. A generalized flow chart for the evaluation of liquefaction hazards to pile supported structures is presented in Figure 4-2. This simplified chart is intended to illustrate the basic procedures involved in evaluating potential liquefaction hazards and developing mitigation programs.

Preliminary Site Investigation A preliminary site evaluation may involve establishing the topography, stratigraphy, and location of the ground water table at the project site. These geologic site evaluations must address the following three basic questions:

a) Are potentially liquefiable soil types present?

b) Are they saturated, and/or may they become saturated at some future date?

c) Are they of sufficient thickness and/or lateral extent as to pose potential risk with respect to major lateral spreading, foundation bearing failure or related settlements, overall site settlements, localized lateral ground movements, or localized ground displacements due to "ground loss"?

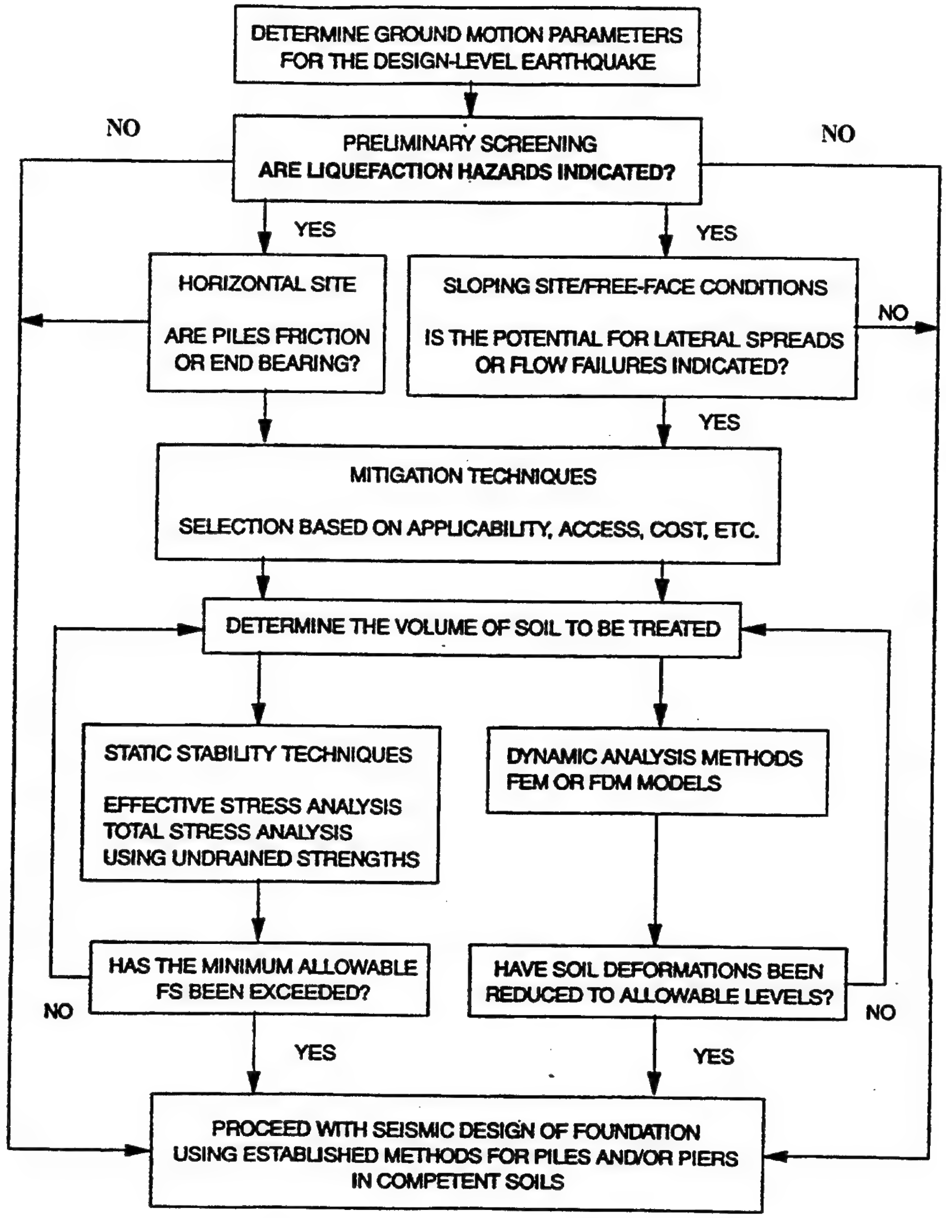

Figure 4-2: A Flow Chart For The Evaluation Of Liquefaction Hazards To Pile Supported Structures

The general geologic information, combined with ground motion data related to the design-level seismic events, can be used to provide a preliminary indication of the potential liquefaction susceptibility of soils at the site. This methodology has been developed by Youd and his co-workers, and is described in a recent state-of-the-art paper on the mapping of earthquake-induced liquefaction (Youd, 1991). In addition, California Department of Conservation, Division of Mines and Geology has established guidelines for mapping areas which might be susceptible to the occurrence of liquefaction. These zones establish where site-specific geotechnical investigations must be conducted to assess liquefaction potential and, if required, provide the technical basis to mitigate the liquefaction hazard. The following is taken directly from their criteria:

Liquefaction Hazard Zones are areas meeting one or more of the following criteria:

1. Areas known to have experienced liquefaction during historic earthquakes. Field studies following past earthquakes indicate liquefaction tends to recur at many sites during successive earthquakes

2. All areas of uncompacted fills containing liquefaction susceptible material that are saturated, nearly saturated, or may be expected to become saturated.

3. Areas where sufficient existing geotechnical data and analyses indicate that the soils are potentially liquefiable. The vast majority of liquefaction hazard areas are underlain by recently deposited sand and/or silty sand. These deposits are not randomly distributed, but occur within a narrow range of sedimentary and hydrologic environments. Geologic criteria for assessing these environments are commonly used to delineate bounds of susceptibility zones evaluated from other criteria, such as geotechnical analysis (Youd, 1991) . Ground water data should be compiled from well logs and geotechnical borings. Analysis of aerial photographs of various vintages may delineate zones of flooding, sediment accumulation, or evidence of historic liquefaction. The Quaternary geology should be mapped and age estimates assigned based on ages reported in the literature, stratigraphic relationships and soil profile descriptions. In many areas of Holocene and Pleistocene deposition, geotechnical and hydrologic data are compiled. Geotechnical investigation reports with Standard Penetration Test (SPT) and/or Cone Penetration Test (CPT) and grain size distribution data can be used for liquefaction resistance evaluations.

4. Areas where geotechnical data are insufficient. The correlation of Seed et al. (1985) , and the $(N_1)_{60}$ data can be used to assess liquefaction susceptibility. Since geotechnical analyses are usually made using limited available data the susceptibly zones should be delineated by use of geologic criteria. Geologic cross sections, tied to boreholes and/or trenches, should be constructed for correlation purposes. The units characterized by geotechnical analyses are correlated with surface and subsurface units and extrapolated for the mapping project.

CDMG criteria uses the minimum level of seismic excitation for liquefaction hazard zones to be that level defined by a magnitude 7.5-weighted peak ground surface acceleration for UBC S2 soil conditions with a 10 percent probability of exceedance over a 50-year period.

In areas of limited or no geotechnical data, susceptibility zones are identified by CDMG geologic criteria as follows:

(a) Areas containing soil deposits of late Holocene age (current river channels and their historic floodplains, marshes and estuaries), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in

50 years is greater than or equal to 0.10 g and the water table is less than 40 feet below the ground surface; or

(b) Areas containing soil deposits of Holocene age (less than 11,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.20 g and the historic high water table is less than or equal to 30 feet below the ground surface; or

(c) Areas containing soil deposits of latest Pleistocene age (between 11,000 years and 15,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.30 g and the historic high water table is less than or equal to 20 feet below the ground surface.

According to CDMG, the Quaternary geology may be taken from existing maps, and hydrologic data should be compiled. Application of this criteria permits development of liquefaction hazard maps which definite regions requiring detailed investigation, allowing concentration of sampling and testing in areas requiring most delineation.

Quantitative Evaluation of Liquefaction Resistance If the results of the preliminary site evaluation indicate that more in-depth studies are warranted, then additional geotechnical characterization of the soils will be necessary. Guidelines for the analysis and mitigation of liquefaction hazards have been presented by the CDMG (Special Publication No. 117). In the context of a factor of safety, the occurrence of liquefaction can be thought of as the capacity of the soil to resist the development of excess pore pressures versus the demand imposed by the seismic ground motions. In practice, the quantitative evaluation of liquefaction resistance is usually based on in-situ Standard Penetration Test (SPT) data (Seed and Idriss, 1982; Seed and De Alba, 1986) and/or Cone Penetration Test (CPT) data (Robertson et al., 1992). The liquefaction resistance of a sand is related to the penetration resistance obtained by either the SPT (N in blows/30 cm (or foot)) or CPT (q_c in kg/cm^2 (or tsf)). The penetration values measured in the field are corrected to account for confining stresses and normalized to obtain a value which corresponds to the N-value that would be measured if the soil was under a vertical effective stress of 1 kg/cm^2 (1 tsf). Additional corrections may be required for SPT N-values depending on the type of drive hammer and release system, length of drill stem, and other factors as outlined by Seed and Harder (1990). The corrected N-value used in the liquefaction analysis is designated $(N_1)_{60}$.

The earthquake-induced cyclic stresses in the soil can be estimated by (a) the simplified evaluation procedure developed by Seed and Idriss (1982), or (b) performing a dynamic soil response analysis. The Seed and Idriss technique yields an estimate of the ratio of the average cyclic shear stress on a horizontal plane in the soil to the initial vertical effective stress on that plane. Correlations of the penetration resistance of soils and cyclic stress ratio (CSR) at sites which did or did not liquefy during recent earthquakes have been established for level ground conditions. In practice, the cyclic stress ratio (CSR) required for the "triggering" of liquefaction

can be determined once the penetration resistance of the soil has been obtained by the use of liquefaction boundary curves, Figure 4-3). The cyclic stress ratio developed during an earthquake is given by the equation:

$$\left(\frac{\tau_{av}}{\sigma_v'}\right) \approx 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \cdot r_{MSF}$$

where τ_{av} is the average cyclic shear stress, σ_v' is the vertical effective stress, σ_v is the total vertical stress, a_{max} is the maximum horizontal ground surface acceleration, g is the acceleration of gravity, r_d is a stress reduction factor which accounts for the fact that the soil column above the soil element behaves as a deformable body, and r_{MSF} is the magnitude scaling factor which is used to convert the CSR required for liquefaction due to a magnitude 7.5 earthquake (the basis for the boundary curves in Figure 4-3 to the magnitude of interest.

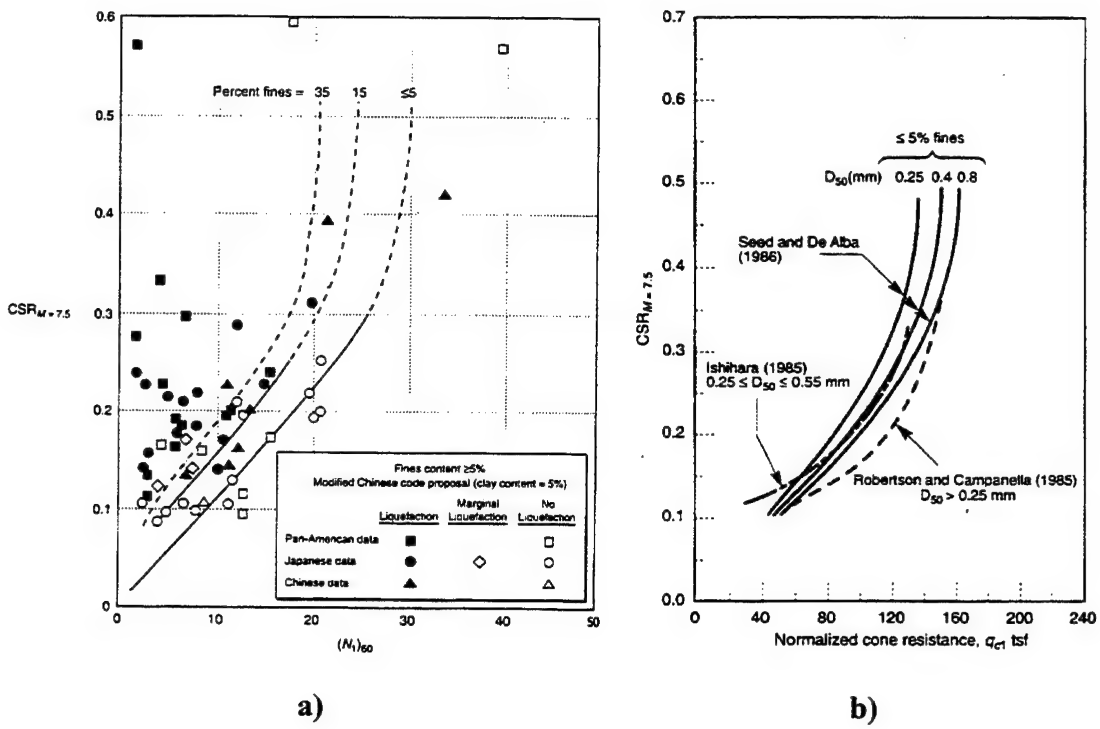

Figure 4-3:Liquefaction Boundary Curves;
A) Seed Et Al., 1975, B) After Mitchell And Tseng, 1990

B)

Several practical points should be noted regarding these plots:

a) The N-values used in the development of the relationship were obtained using an ASTM standard sampler driven by a 64 kg (140 lb) weight falling 76 cm (30 in). In light of the variety of soil samplers and driving mechanisms (e.g.; safety hammers, donut hammers, slip-jars, etc.) commonly used in practice, the engineer should realize that correlations between N-values obtained by various methods are tenuous at best. Appropriate caution should be exercised when interpreting potential liquefaction behavior based on N-values from non-standard techniques.

b) The boundaries between liquefaction and non-liquefaction are based on case histories for the surface-evidence of liquefaction. It is possible that sites classified as not exhibiting liquefaction experienced the development of significant excess pore pressures that were not manifested at the surface. The boundaries are therefore not intended to delimit the definitive occurrence or nonoccurrence of liquefaction, but rather an approximate indication of whether ground failures may be experienced.

c) The intensity of the ground motions are accounted for in the formulation of the CSR. The number of load cycles is also an important parameter. To represent the effects of the number of cycles, a magnitude scaling factor has been included. Recent studies have served to enhance the magnitude scaling factors originally derived by Seed and Idriss (1982). The results of these studies are presented in Figure 4-4 and indicate that the current scaling factors are overconservative at earthquake magnitudes less than roughly 7.0 (Arango, 1996).

d) This plot is appropriate for horizontal sites only. Approximate corrections can be made to the cyclic stress ratio required to cause liquefaction (CSR_l) for sloping sites (Seed and Harder, 1990).

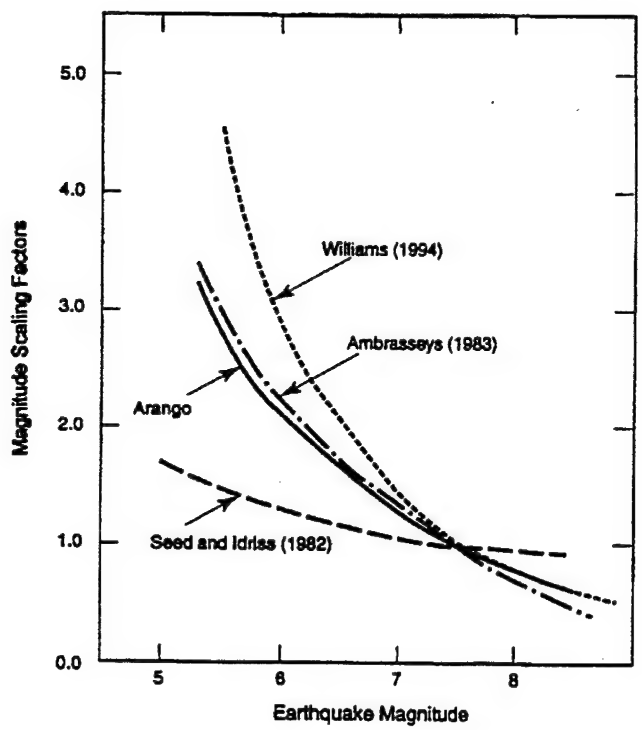

Figure 4-4: Comparison of Earthquake Magnitude Scaling Factors from Various Sources (Arango, 1996)

e) The methods for evaluating the liquefaction resistance of soils have been developed for free-field conditions at horizontal sites where there are no static shear stresses on horizontal planes. This technique is applicable for sites of new construction, yet liquefaction hazard

studies may also be warranted for existing facilities. Foundation loads imposed by structures (particularly around the perimeter of the foundation) induce static shear stresses on horizontal planes in the soil. These additional stresses will affect the liquefaction susceptibility of the soil, increasing the hazard in loose soils (relative density ≤ 40%) and potentially decreasing the liquefaction susceptibility in medium dense and dense soils (relative density ≥ 50%). The influence of foundation stresses on the liquefaction behavior of soils has been investigated by Rollins and Seed (1990).

The simplified N-based method of liquefaction hazard evaluation has been shown to provide reasonable estimates for the occurrence of liquefaction during numerous recent earthquakes. This agreement with the field case histories has led to its widespread use in engineering practice.

If the Seed and Idriss method clearly demonstrates the absence of liquefaction hazard, then this investigation may be sufficient. However, if the occurrence of liquefaction is predicted, additional seismic hazards should be addressed. These include:

- Hazards associated with lenses of liquefiable soil or by potentially liquefiable layers which underlie resistant, nonliquefiable capping layers. In situations where few, thin lenses of liquefiable soil are identified, the interlayering of liquefiable and resistant soils may serve to minimize structural damage to light, ductile structures. It may be determined that "life safety" and/or "serviceability" requirements may be met despite the existence of potentially liquefiable layers. Ishihara (1985) developed an empirical relation which provides approximate boundaries for liquefaction-induced surface damage for soil profiles consisting of a liquefiable layer overlain by a resistant, or protective, surface layer, Figure 4-5. This relation has been validated by Youd and Garris (1995) for earthquakes with magnitudes between 5.3 and 8. In light of the heterogeneous nature of most soil deposits and the uncertainties inherent in the estimation of ground motion parameters, it is recommended that this method of evaluation be considered for noncritical structures only.

- The potential for lateral ground movements and the effects on foundations and buried structures (Bartlett and Youd, 1995).

- The effects of changes in lateral earth pressures on retaining structures (Ebeling and Morrison, 1993; Power et al., 1986).

- Estimated total and differential settlements at the ground surface due to liquefaction and subsequent densification of the soils (Ishihara and Yoshimine, 1992; Tokimatsu and Seed, 1987).

Several of these liquefaction hazards are addressed in the following section.

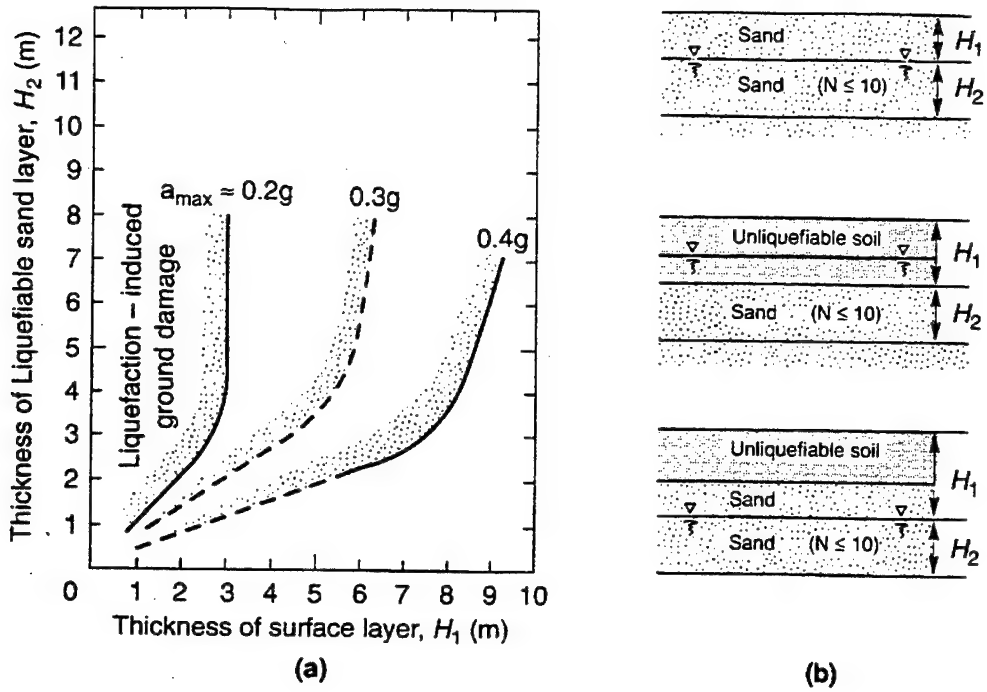

Figure 4-5: (A) Relationship between thickness of liquefiable layer and thickness of overlying layer at sites for which surface manifestation of liquefaction has been observed, and (B) guides to evaluation of respective layer thicknesses (after Ishihara, 1985)

Post-Liquefaction Behavior of Sandy Soils

Post-Liquefaction Volume Change of Sandy Soils The densification of partially-saturated or saturated loose sandy soils due to cyclic loading can result in damaging differential settlement. This phenomena was graphically demonstrated at Port Island (Port of Kobe) after the 1995 Hyogoken Nanbu earthquake where settlements over much of the island averaged 50 cm, with maximum settlements of over 1 m in many places. Several methods have been developed for estimating the magnitude of earthquake-induced settlements in sandy soils (e.g., Ishihara and Yoshimine, 1992; Tokimatsu and Seed, 1987). A simple chart for estimating the volumetric strain in sandy soils during earthquakes is shown in Figure 4-6..

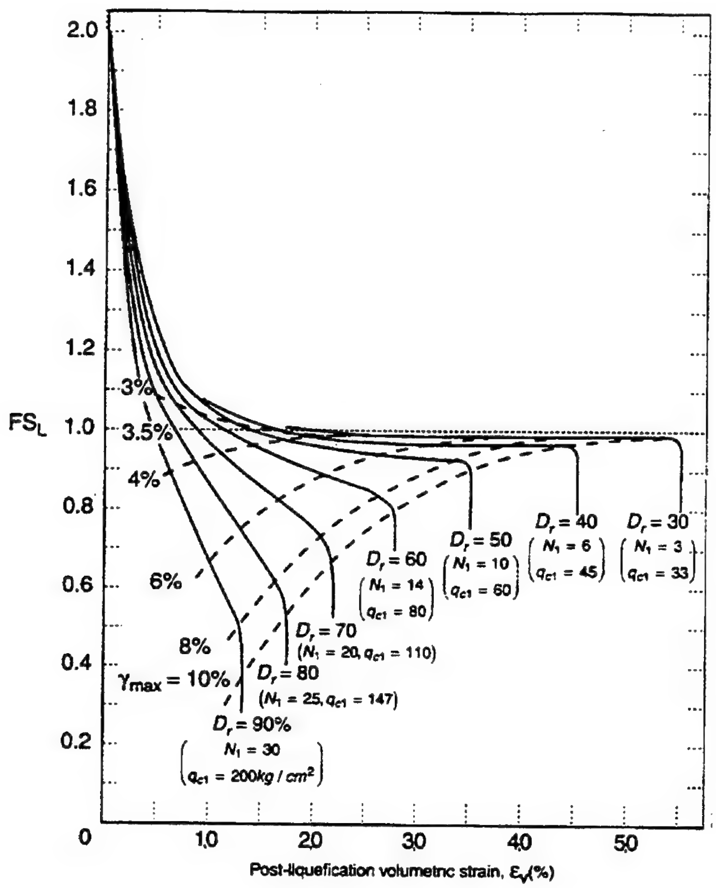

Figure 4-6: Post Volumetric Shear Strain for Clean Sands (Ishihara and Yoshimine, 1992)

Post-Liquefaction Shear Strength In order to evaluate the stability of slopes and embankments, waterfront retaining structures, and other structures underlain by liquefied soils, the strength of the liquefied material must be estimated. Although the condition of initial liquefaction is often defined as the state at which the effective stress (and therefore the shear strength) is equal to zero, the soil will mobilize a residual shear strength if it undergoes large shear strains. In two recent studies, the undrained shear strength of the liquefied sand has been back-calculated from a number of documented slope failures (Seed and Harder, 1990; Stark and Mesri, 1992). These reports describe methods for estimating the residual undrained shear strength of sands based on the SPT penetration resistance of the soil prior to the earthquake, Figure 4-7. The undrained strength values obtained from these relations are used in standard total stress stability analyses equivalent to those commonly performed when evaluating the short term stability of embankments or foundations on saturated clayey soils.

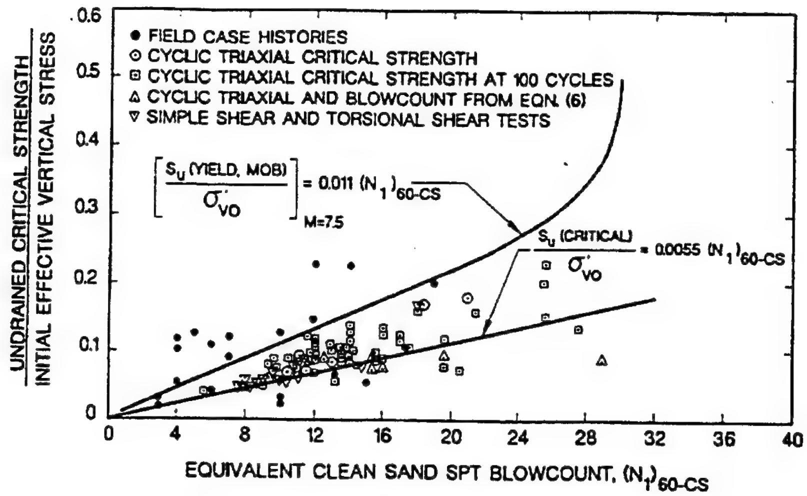

Figure 4-7. Relationship between Undrained Critical Strength Ratio and Equivalent Clean Sand Blow Count (Stark and Mesri, 1992)

Liquefaction-Induced Ground Failures The use of the Seed and Idriss method of evaluating liquefaction hazard constitutes what may be termed a "triggering" analysis. This term is used to indicate that the method identifies the CSR required for the surface manifestation of liquefaction. Soils beneath slopes of as little as 0.2% may experience flow failures subsequent to the onset of initial liquefaction. In order to evaluate the seismic performance of pile supported structures, breakwaters, pipelines and other structures near slopes, it is necessary to estimate the lateral deformation that is likely to occur during the design level earthquakes. Both empirical and numerical techniques have been developed for this analysis. Youd and his co-workers have developed simplified procedures for estimating the magnitude of lateral displacements based on data from numerous case histories (Youd and Perkins, 1987; Bartlett and Youd, 1995). A regression analysis of field data has resulted in the development of an equation for predicting the lateral deformation for free-field sites (i.e., in the absence of piles and structures) on gentle slopes or adjacent to free-faces such as stream banks or dredged channels.

The empirical techniques have been augmented by the results of several numerical studies. The numerical studies are largely based on the "sliding block" technique (described in Section 4.6) wherein coherent blocks of soil are modeled as moving over a liquefied layer. In this case the undrained residual (or "steady state") strength will control the behavior of the soil mass. The seismic stability of slopes underlain by potentially liquefiable soils can be assessed using estimated values for the undrained shear strength of the sandy soils.

The transition from the static shear strength to the undrained residual strength requires several cycles of loading, and it would be advantageous to account for this progressive strength loss in ground response analyses. Also, the flow behavior of the soil and subsequent reconsolidation should, theoretically, be modeled in analyses involving soil liquefaction. In light of this stress-strain-strength behavior, the phenomena of liquefaction-induced ground failure is complex, and the numerical models used for the evaluation of this hazard have relied on simplifying assumptions. The sliding block method has been used in conjunction with residual

undrained strengths for sands to predict the seismic performance of slopes and earth structures (Baziar et al., 1992; Byrne et al., 1994).

Code Provisions and Factors Of Safety Against Liquefaction

In general building codes do not give extensive guidance for liquefaction apart for the need for investigating a site for geologic hazards. The AASHTO Standard Specification For Highway Bridges (1992) suggests the factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in cases of important bridge sites. While not specifically stated it is presumed that this is to be used in conjunction with their acceleration maps which give a 10 percent probability of exceedance in 50 years. Similar recommendations have been provided in the CDMG Guidelines for Evaluation and Mitigating Seismic Hazards in California (1997). The geotechnical panel assembled for the preparation of the CDMG report recommended that "If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary."

Techniques for Mitigating Liquefaction Hazards

If hazard evaluations indicate that there are potentially liquefiable soils of such extent and location that an unacceptable level of risk is presented, then soil improvement methods should be considered for the mitigation of these hazards. Mitigation must provide suitable levels of protection with regard to the three general types of liquefaction hazard previously noted: (a) potential global translational site instability; (b) more localized problems; and (c) failure of retaining structures.

Liquefaction remediation must address the specifics of the problem on a case by case basis. These specifics include the local site conditions, the type of structure, and the potential for flows and settlements. When liquefaction occurs, there can be a potential for extensive lateral flow slides which can affect a large area, a global site instability. Also there can be local soil settlements and bearing failures which affect a structure on a local level. Specific types of structures can have specific associated problems. Buried structures can become buoyant. Retaining structures where the backfill has liquefied can experience increased lateral loading and deformation.

Potentially suitable methods of mitigation may include the following: removal and replacement, dewatering, in-situ soil improvement, containment or encapsulation structures, modification of site geometry, deep foundations, structural systems and, if possible, alternate site selection. In general, soil improvement methods reduce the liquefaction susceptibility of sandy soils by increasing the relative density, providing conduits for the dissipation of excess pore pressures generated during earthquakes, and/or providing a cohesive strength to the soil. The effectiveness and economy of any method, or combination of methods, will depend on geologic and hydrologic factors (e.g.; soil stratigraphy, degree of saturation, location of ground water table, depth of improvement, volume of soil to be improved, etc.) as well as site factors (e.g.; accessibility, proximity to existing structures, etc.). An overview of the available liquefaction remediation measures is provided in Table 4-1.

Significant advances have been made in the field of soil improvement over the past several decades. These developments have been made on two fronts: (a) an increased understanding of geotechnical hazards; and (b) the development of innovative construction techniques by specialty contractors. Practical information which addresses recent advances in the mitigation of liquefaction hazards to foundations can be found in numerous recent publications (Borden et al., 1992; Hryciw, 1995; Kramer and Siddharthan, 1995). Several techniques of improvement for liquefiable soils referenced in the geotechnical literature include: (a) densification (e.g.; vibro-methods, dynamic compaction, deep blast

Table 4-1. Liquefaction Remediation Measures (after Ferritto, 1997B)

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
1) Vibratory Probe a) Terraprobe b) Vibrorods c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); > 30 m sometimes; vibrowing, 40 m.	Moderate
2) Vibrocompaction a) Vibrofloat b) Vibro-Composer system.	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	> 20 m	Low to moderate
3) Compaction Piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	> 20 m	Moderate to high
4) Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best, other types can also be improved.	30 m (possibly deeper)	Low
5) Displacement (compaction grout)	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
6) Surcharge/buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	Dependent on size of surcharge/buttress	Moderate if vertical drains are used
7) Drains a) Gravel b) Sand c) Wick d) Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand. silt, clay.	Gravel and sand > 30 m; depth limited by vibratory equipment; wick, > 45 m	Moderate to high
8) Particulate grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout methods
9) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser.	Unlimited	High

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
10) Pressure injected lime	Penetration grouting-fill soil pores with lime	Medium to coarse sand and gravel.	Unlimited	Low
11) Electrokinetic injection	Stabilizing chemical moved into and fills soil pores by electro-osmosis or colloids in to pores by electrphoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive
12) Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
13) Mix-in-place piles and walls	Lime, cement or asphalt introduced through rotating auger or special in-place mixer.	Sand, silts, clays, all soft or loose inorganic soils.	> 20 m (60 m obtained in Japan)	High
14) Vibro-replacement stone and sand columns a) Grouted b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	> 30 m (limited by vibratory equipment)	Moderate
15) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high
16) Blasting	Shock waves an vibrations cause limited liquefaction, displacement, remolding, and settlement to higher density.	Saturated, clean sand; partly saturated sands and silts after flooding.	> 40 m	Low

densification, compaction grouting, and compaction piles); (b) drainage to allow rapid dissipation of excess pore pressures (e.g.; vibro-replacement and stone columns); and (c) chemical modification/cementation (e.g.; permeation grouting, jet grouting, and deep mixing). Additional possible methods of increasing the liquefaction resistance of soils include permanent dewatering, and removal of loose soils and replacement at a suitable compactive effort.

The most common methods for remediation of liquefaction hazards at open, undeveloped sites include (in order of increasing cost): (a) deep dynamic compaction; (b) vibro-compaction; (c) vibro-replacement, excavation and replacement; and (c) grouting methods. Each of these techniques results in a significant displacement of soil. On projects involving foundation remediation adjacent to existing structures or buried utilities, ground movements must be minimized to avoid architectural and structural damage. Several projects with these constraints have utilized grouting techniques to stabilize potentially liquefiable soils.

Several methods of densification have been used including vibroprobe, vibro-compaction, dynamic compaction, compaction grouting, and compaction piles. Substitution or replacement of soil to improve drainage has been used including vibro-replacement and stone columns. Techniques like stone columns achieve their effectiveness by replacing liquefiable cohesionless soils with stiffer columns of gravel and rock which improves strength and promotes drainage. Cement grouting, jet grouting and deep mixing have been used as chemical means of eliminating/reducing liquefaction potential. Surcharging a site increases liquefaction resistance

by increasing the effective confining pressures. Table 4 presents a summary of methods used for remediation and their relative cost as reported by Professor Whitman (NRC 1985). Navy facilities on Treasure Island during the 1989 Loma Prieta earthquake can attest to the effectiveness of remediation. Areas where remediation was done performed well while other areas suffered settlements of 6 to 8 inches and lateral spreads. Observation of damage during the 1995 Hyogoken Nanbu (Kobe) Earthquake again confirmed the performance of improved sites. Preloading, sand drains, sand compaction piles, and vibro-compaction were shown to be effective.

Method	Vertical Settlement	
	Range (cm)	Average (cm)
Untreated	25 to 95	42
Preloading	15 to 60	30
Sand drains	0 to 40	15
Sand drains & preloading	0 to 25	12
Vibro-compaction	0 to 5	near 0
Sand compaction piles	0 to 5	near 0

Generally costs increase from dynamic compaction to vibro-compaction to replacement The measure of effectiveness of a remediation undertaking is the increase in minimum soil density and specifications usually measure this by the improvement in penetration resistance or laboratory testing. Engineering practice tends to be conservative and factors of safety from 1.5 to 2.0 against liquefaction are often specified. These values may be harder to achieve at the waterfront in regions of high seismicity.

The contract specifications for ground improvement must include a program for the verification of soil improvement. Ground improvement specifications typically require that a minimum soil density (as related to a penetration resistance) be achieved. A level of risk can be assessed by the factor of safety against liquefaction -- which is defined as the ratio of the CSR required for liquefaction of the improved soil to the CSR induced by the design earthquake. Factors of safety between 1.5 and 2.0 are commonly specified for non-critical structures. It should be noted that the penetration resistance of recently deposited, or modified, soils increases with time after treatment. This phenomena, termed "aging" must be accounted for when specifying the time between soil improvement and the in-situ verification testing. There is also evidence that the SPT and CPT tests are not sufficiently sensitive to detect the minor changes in soil fabric that can significantly increase the liquefaction resistance of the soil. This has led to the incorporation of lab testing programs in addition to field tests for verification of soil improvement for liquefaction hazards.

There are currently very few references on the volume of soil that should be improved to mitigate liquefaction hazards to structures (JPHRI, 1997; Dickenson and McCullough, 1998). At horizontal sites, general recommendations for buildings and pile-supported structures may call

for soil improvement to the base of the liquefiable deposit (or to a maximum depth of 15 to 18 meters) and over a lateral extent equal to the thickness of the layer, plus an additional increment based on judgment. The uncertainty associated with general recommendations like these is compounded for sloping sites and areas adjacent to waterfront retaining structures. In these situations, the volume of soil that may be involved in a lateral spread is difficult to ascertain, therefore existing recommendations tend to be very conservative. Based on shake table tests, Iai (1992) has proposed tentative guidelines for the extent of soil improvement required to mitigate liquefaction hazards behind caissons. These recommendations have been incorporated into the guidelines for soil improvement adjacent to waterfront retaining structures developed by the Japan Port and Harbor Research Institute (JPHRI, 1997). Several examples for soil improvement adjacent to waterfront retaining structures are contained in Figure 4-8.

Very few case studies exist for the seismic performance of improved soil sites. The effectiveness of the soil improvement at limiting deformations adjacent to foundations and retaining structures will be a function of the strength and duration of shaking. In only a limited number of cases have the improved sites been subjected to design-level ground motions. The cases that have been documented demonstrate a substantial reduction in liquefaction-induced ground failures and ground deformations due to the ground treatment (e.g., Iai et al., 1994; Ohsaki, 1970; Mitchell et al., 1995; Yasuda et al., 1996). However, as noted in several of these papers, the ground deformations were not reduced to imperceptible levels. This observation is especially germane when establishing allowable deformation limits for waterfront retaining structures, where adjacent gantry cranes and other sensitive components may be damaged by lateral movements of the walls. In several cases (e.g., flexible retaining structures such as anchored sheet pile bulkheads and cellular sheet pile bulkheads), these structures may deform when subjected to strong ground motions despite the utilization of ground treatment. In cases such as this, the ground treatment would serve to preclude catastrophic failure of the retaining structures.

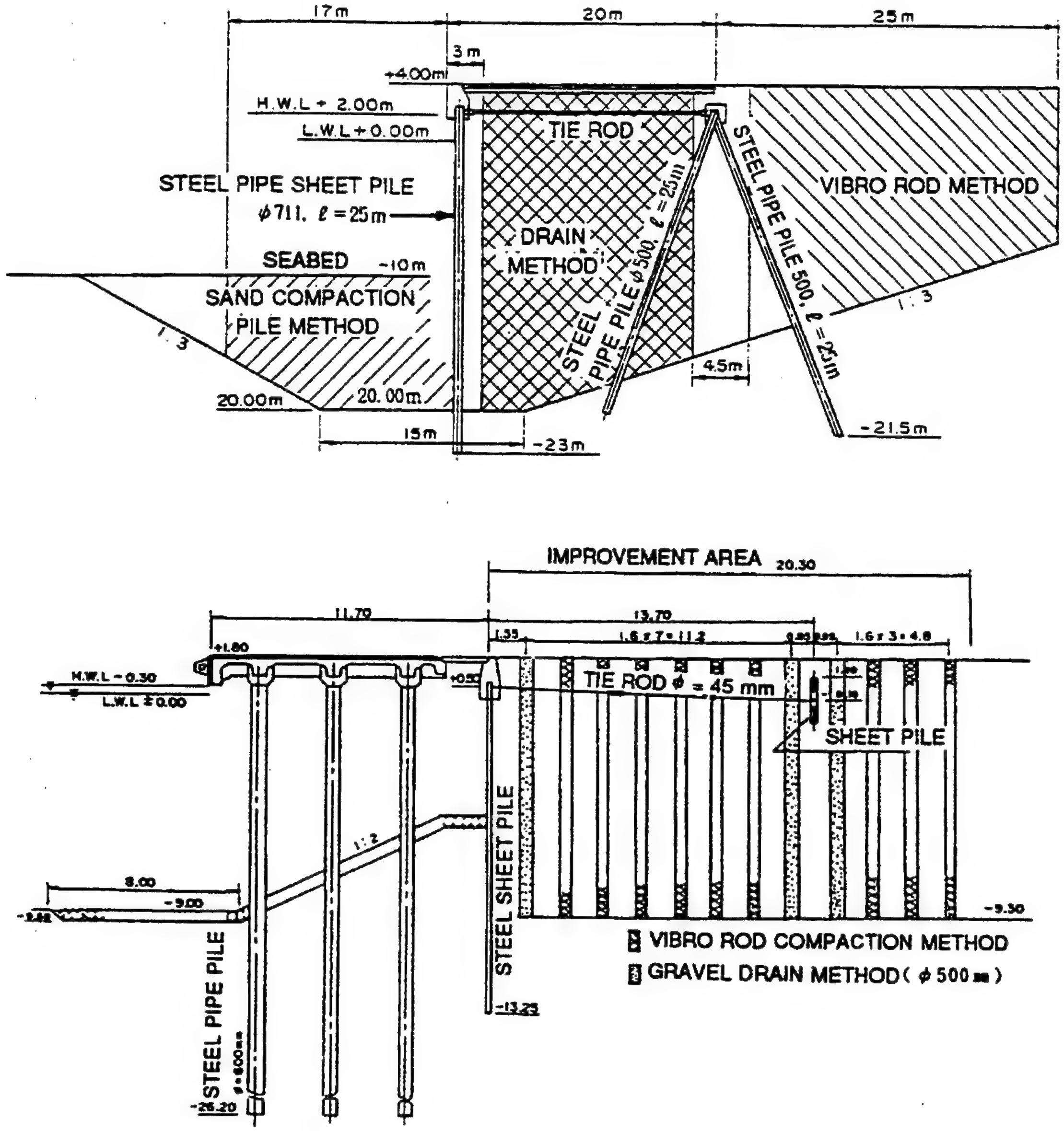

Figure 4-8. Examples of Soil Improvement for Waterfront Retaining Structures (PHRI, 1997)

One particularly relevant case study involving the waterfront retaining structures at the Kushiro Port in Japan has been documented by Iai and his co-workers at the Port and Harbour Research Institute (Iai et al., 1994). Kushiro Port is located on the east coast of the island of Hokkaido, a region that is prone to large subduction zone earthquakes. Prior to 1993, the port had been subjected to at least two damaging earthquakes (the M_{JMA} 8.1 1952 Tokachi-Oki Earthquake and the M_{JMA} 7.4 1973 Nemuro-Hanto-Oki Earthquake) and these experiences appear to have influenced the seismic design criteria subsequently adopted at the port. In the older portions of the port, the sandy backfill soil adjacent to retaining structures had been left in its original loose state while, the in the newer sections of the port (constructed as late as 1992), a soil improvement program was implemented to reduce the liquefaction susceptibility of the fills near retaining

walls. The remediation project called for the use of various types and degrees of soil improvement in the waterfront. In 1993, the port was subjected to strong ground motions generated during the M_{JMA} 7.8 Kushiro-Oki Earthquake. It is significant to note that while the waterfront areas at which soil improvement was implemented performed successfully, thereby allowing the port to continue operation immediately after the earthquake, the waterfront retaining structures founded in loose soils experienced dramatic liquefaction-induced failures.

Slope Instability Hazards

Introduction

Sloping ground conditions exist throughout ports as natural and engineered embankments (e.g., river levees, sand or rock dikes, etc., and dredged channel slopes). On-shore and submarine slopes at ports have been found to be vulnerable to earthquake induced deformations. High water levels and weak foundation soils common at most ports can result in slopes which have marginal static stability and which are very susceptible to earthquake induced failures. In addition to waterfront slopes, several recent cases involving failures of steep, natural slopes along marine terraces located in backland areas have resulted in damage to coastal ports. Large scale deformations of these slopes can impede shipping and damage adjacent foundations and buried structures thereby limiting port operations following earthquakes.

The most commonly used methods for analysis of slope stability under both static and dynamic conditions are based on standard rigid body mechanics and limit equilibrium concepts that are familiar to most engineers. The development and application of these techniques are introduced in most geotechnical engineering textbooks and in numerous design manuals. Therefore, they will not be described here. Instead, this section will introduce the strengths and limitations of these analysis techniques as applied at port facilities.

Pseudostatic Methods of Analysis

Standard, limit equilibrium methods for analyzing the static stability of slopes are routinely used in engineering practice. The use of these design tools have several advantages in practice: (a) the techniques are familiar to most engineers; (b) requisite input includes standard geotechnical parameters that are obtained during routine foundation investigations; and (c) the methods have been coded in very straightforward, efficient computer programs that allow for sensitivity studies to be made of various design options.

For use in determining the seismic stability of slopes, limit equilibrium analyses are modified slightly with the addition of a permanent lateral body force which is the product of a *seismic coefficient* and the mass of the soil bounded by the potential slip. The seismic coefficient (usually designated as k_h, N_h) is specified as a fraction of the peak horizontal acceleration, due to the fact that the lateral inertial force is applied for only a short time interval during transient earthquake loading. Seismic coefficients are commonly specified as roughly $^1/_3$ to $^1/_2$ of the peak horizontal acceleration value (Seed, 1979; Marcuson, et al., 1992).

In most cases involving soils which do not exhibit considerable strength loss after the peak strength has been mobilized, common pseudostatic rigid body methods of evaluation will generally suffice for evaluating the stability of slopes. These methods of evaluation are well established in the technical literature (Kramer, 1996). Although these methods are useful for indicating an approximate level of seismic stability in terms of a factor of safety against failure, they suffer from several potentially important limitations. The primary disadvantages of pseudostatic methods include: (a) they do not indicate the range of slope deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils is incorporated in only a very simplified, "decoupled" manner; (c) progressive deformations that may result due to cyclic loading at stresses less than those required to reduce specific factors of safety to unity are not modeled; (d) strain softening behavior for liquefiable soils or sensitive clays is not directly accounted for; and (e) important aspects of soil-structure interaction are not evaluated.

Limited Deformation Analysis

In most applications involving waterfront slopes and embankments, it is necessary to estimate the permanent slope deformations that may occur in response to the cyclic loading. Allowable deformation limits for slopes will reflect the sensitivity of adjacent structures, foundations and other facilities to these soil movements. Enhancements to traditional pseudostatic limit equilibrium methods of embankment analysis have been developed to estimate embankment deformations for soils which do not lose appreciable strength during earthquake shaking (Ambraseys and Menu, 1988; Makdisi and Seed, 1978; Jibson, 1993).

Rigid body, "sliding block" analyses, which assume the that soil behaves as a rigid, perfectly plastic material, can be used to estimate limited earthquake-induced deformations. The technique, developed by Newmark (1965) and schematically illustrated in Figure 4-9, is based on simple limit equilibrium stability analysis for determining the critical, or yield, acceleration which is required to bring the factor of safety against sliding for a specified block of soil to unity. The second step involves the introduction of an acceleration time history. When the ground

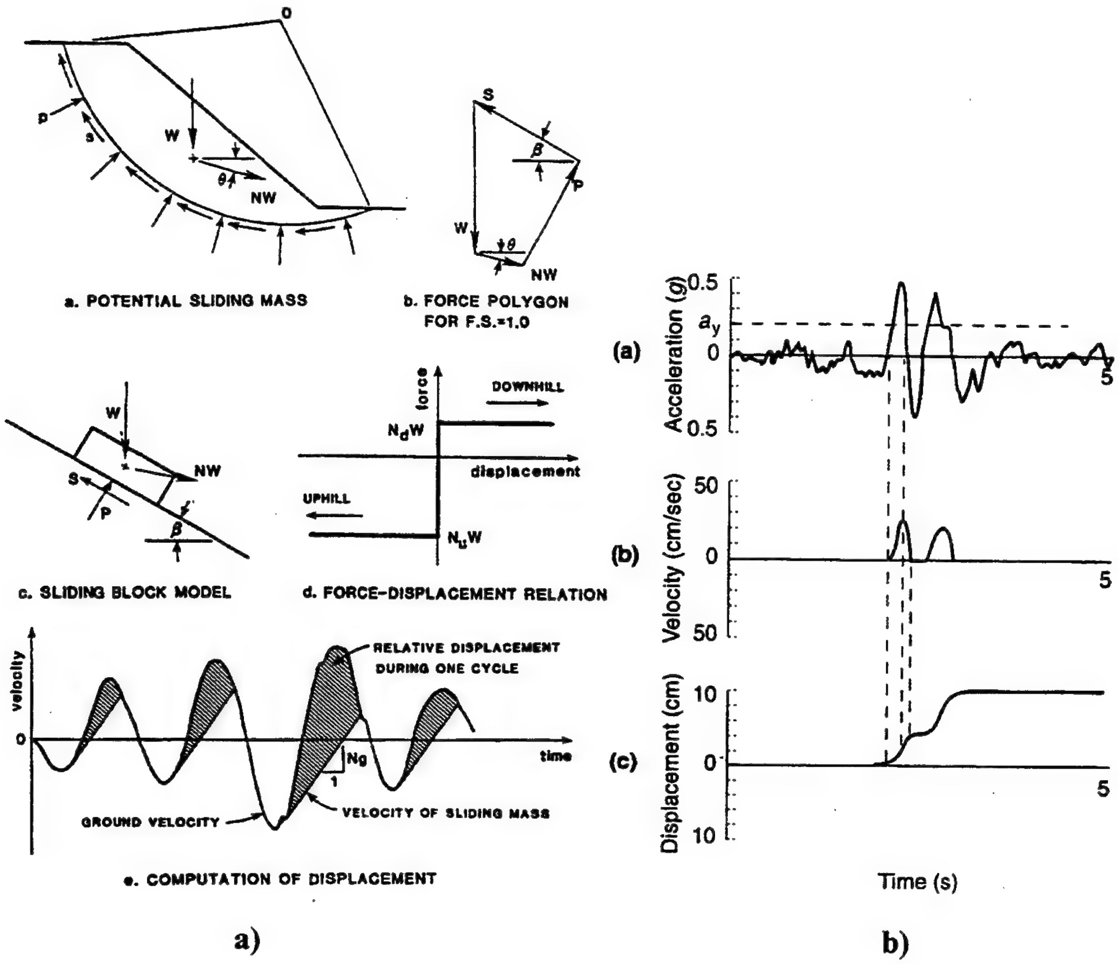

Figure 4-9. Elements of Sliding Block Analysis, A) Hynes-Griffin and Franklin, 1984, B) after Wilson and Keefer, 1985

motion acceleration exceeds the critical acceleration (a_{crit}, a_y) the block begins to move down slope. By double integrating the area of the acceleration time history that exceeds a_{crit}, the relative displacement of the block is determined. A simple spreadsheet routine can be used to perform this calculation (Jibson, 1993). Numerical studies based on this method of analysis have lead to the development of useful relationships between ground motion intensity and the seismically-induced deformations (e.g. Ambraseys and Menu, 1988; Makdisi and Seed, 1978; Jibson, 1993). The relationship proposed by Makdisi and Seed is shown in Figure 4-10.

The amount of permanent displacement depends on the maximum magnitude and duration of the earthquake. The ratio of maximum acceleration to yield acceleration of 2.0 will result in block displacements of the order of a few inches for a magnitude 6 1/2 earthquake and several feet for a magnitude 8 earthquake. It should be noted that significant pore pressure increases may be induced by earthquake loading in saturated silts and sands. For these soils a potential exists for a significant strength loss. For dense saturated sand, significant undrained shear strength can still be mobilized even when residual pore pressure is high. For loose sands, the residual undrained strength which can be mobilized after high pore pressure build-up is very low and is often less than the static undrained shear strength. This may result in flow slides or large ground deformations.

Given that the sliding block analyses are based on limit equilibrium techniques, they suffer from many of the same deficiencies previously noted for pseudostatic analyses. One of the primary limitations with respect to their application for submarine slopes in weak

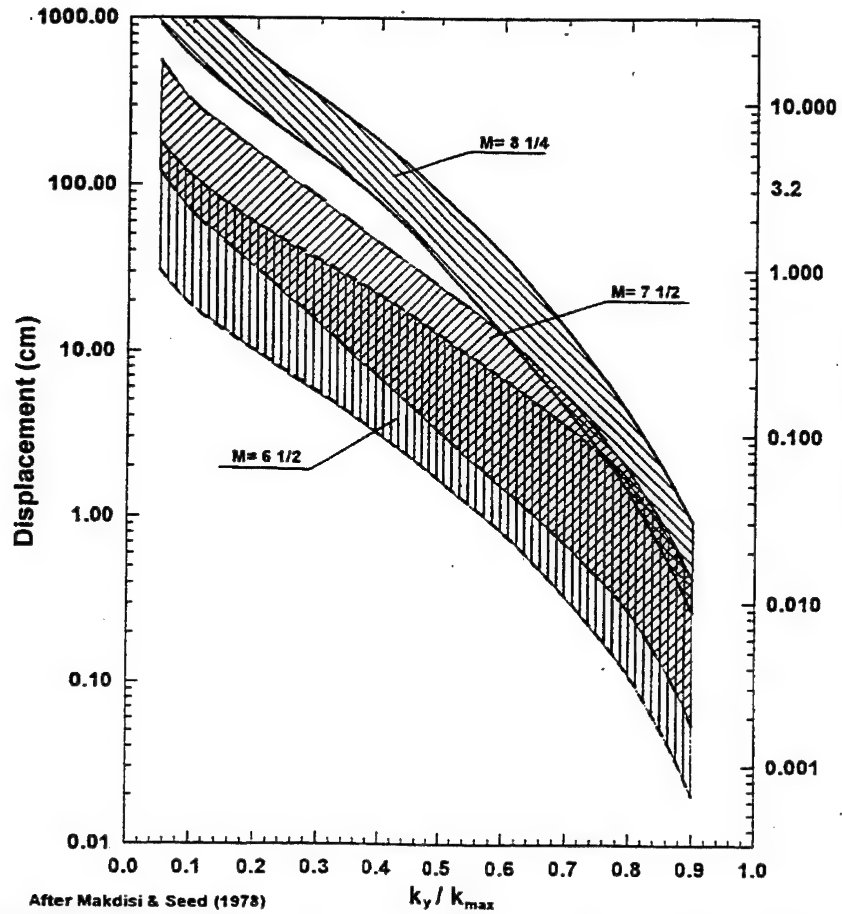

Figure 4-10. Empirical Relationships between Permanent Displacement of sliding Block and ratios of accelerations (after Makdisi and Seed, 1978)

soils is that strain softening behavior is not directly accounted incorporated in the analysis. The sliding block methods have, however, been applied for liquefiable soils by using the post-liquefaction undrained strengths for sandy soils. A recent method proposed by Byrne et al. (1994) has been developed for contractive, collapsible soils which are prone to liquefaction.

Advanced Numerical Modeling of Slopes

In situations where the movement of a slop impacts adjacent structures, such as pile supported structures embedded in dikes, buried lifelines and other soil-structure interaction problems, it is becoming more common to rely on numerical modeling methods to estimate the range of slope deformations which may be induced by design level ground motions (Finn, 1990). The numerical models used for soil-structure interaction problems can be broadly classified

based on the techniques that are used to account for the deformations of both the soil and the affected structural element. In many cases the movement of the soil is first computed, then the response of the structure to these deformations is determined. This type of analysis is termed *uncoupled*, in that the computed soil deformations are not affected by the existence of embedded structural components. A common enhancement to this type of uncoupled analysis includes the introduction of an iterative solution scheme which modifies the soil deformations based on the response of the structure so that compatible strains are computed. An example of this type of analysis is drag loading on piles due to lateral spreading. In an uncoupled analysis the ground deformations would be estimated using either an empirical relationship (e.g. Bartlett and Youd 1995) or a sliding block type evaluation (e.g. Byrne et al., 1994) as discussed in Section 0 and 0. Once the pattern of ground deformations has been established a model such as LPILE (Reese and Wang, 1994) can be used to determine the loads in the deformed piles. In addition, modifications can be made to the p-y curves to account for the reduced stiffness of the liquefied soil (O'Rourke and Meyerson, 1994). The lateral spread displacement is forced onto the p-y spring (i.e., drag loading).

In a *coupled* type of numerical analysis the deformations of the soil and structural elements are solved concurrently. Two-dimensional numerical models such as FLUSH (Lysmer et al., 1975), FLAC (Itasca, 1995), DYSAC2 (Muraleetharan et al., 1988), and LINOS (Bardet, 1990) have been used to model the seismic performance of waterfront components at ports (e.g., Finn, 1990; Roth et al., 1992; Werner, 1986; Wittkop, 1994). The primary differences in these numerical analyses include; (a) the numerical formulation employed (e.g., FEM, FDM, BEM), (b) the constitutive model used for the soils, and (c) the ability to model large, permanent deformations. Each of the methods listed have been useful in evaluating various aspects of dynamic soil-structure interaction.

Advanced numerical modeling techniques are recommended for soil-structure interaction applications, such as estimating permanent displacements of slopes and embankments with pile supported wharves. The primary advantages of these models include: (a) complex embankment geometries can be evaluated, (b) sensitivity studies can be made to determine the influence of various parameters on the seismic stability of the structure, (c) dynamic soil behavior is much more realistically reproduced, (d) coupled analyses which allow for such factors as excess pore pressure generation in contractive soils during ground shaking and the associated reduction of soil stiffness and strength can be used, (e) soil-structure interaction and permanent deformations can be evaluated. Disadvantages of the numerical analyses methods include: (a) the engineering time required to construct the numerical model can be extensive, (b) numerous soil parameters are often required, thereby increasing laboratory testing costs (the number of soil properties required is a function of the constitutive soil model employed in the model), (c) very few of the available models have been validated with well documented case studies of the seismic performance of actual retaining structures, therefore the level of uncertainty in the analysis is often unknown.

Mitigation of Seismic Hazards Associated with Slope Stability

Remedial strategies for improving the stability of slopes have been well developed for both onshore and submarine slopes. Common techniques for stabilizing slopes include: (a)

modifying the geometry of the slope; (b) utilization of berms; (c) soil replacement (key trenches with engineered fill); (d) soil improvement; and (e) structural techniques such as the installation of piles adjacent to the toe of the slope. Constraints imposed by existing structures and facilities, and shipping access will often dictate which of the methods, or combinations of methods, are used.

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CHAPTER 5 SUPPORTING LIFELINES

Lifelines are key facilities and utility systems which are vital to the operation of a terminal. They include fire detection and suppression, electric power, gas and liquid fuels, telecommunications, transportation and water supply and sewers. The following explains the development of the proposed design criteria and gives current procedures and all relevant codes. Observed damage from previous earthquakes was analyzed to develop failure modes, from which a design criteria was produced.

Potential problems facing a terminal after an earthquake are building structural failures, damage to waterfront retaining structures, tank failures, crane failures, utilities disruption and hazardous materials spills. Typical lifeline problems involve above ground and underground pipeline breaks from soil movement, collapse of pipelines caused by failed supports, shifting of tanks on their foundations, and buckling of tanks. Related factors which add to the complexity of recovery are the dislodging of asbestos or encapsulated asbestos insulation; industrial equipment damage caused by sliding or overturning, or internal failures; falling containers of hazardous materials which may rupture and impede recovery. Other factors can complicate the ability to respond to these releases, including: lack of water for washing down spills, disruption in communication, closure of roads, and lack of transportation access routes.

While guidance can only be given in a general form since specific circumstances control each case, the Resource Conservation and Recovery Act as a public law establishes mandates concerning the pollution of the environment and as such has direct relevance to this criteria. It sets a high seismic design level at which municipal waste facilities are to function to preclude contamination of the environment. This law requires that we place a high value on ground water and preclude contamination. As such this is probably the controlling relevant guidance for non-nuclear polluting or hazardous materials.

Essential Vs. Ordinary Construction

When considering a facility supporting an essential function, it is critical that the facility be considered as a system. It is not sufficient to consider a facility simply as a building structure, but rather it is required to consider all the elements required to accomplish the mission to be accomplished in that structure. This usually includes requirements for fire detection and suppression, electrical power, mechanical systems, water and sewer, communications, road access etc.

Seismic Codes Related To Lifelines

Current seismic codes when viewed as an ensemble, form a basis for understanding the state-of-the-art of risk quantification and the engineering profession's determination of prudent action. This section summarizes a number of seismic standards. DOE procedures and The Resource Conservation and Recovery Act (40 CFR 248 -USEPA 1991) both directly consider hazardous materials.[1]

1994 Uniform Building Code- The building code has been one of the origins for lifeline design under the category of non-building structures. The pseudostatic approach calculates an equivalent lateral force, V as;

$$V = [\ (ZIC) / R_w\ ]^* \ W$$

$$C = 1.25\ S / T^{2/3}$$

where

T	Structural period < 2.75 seconds
Z	Zone factor
I	Importance factor = 1.0 to 1.25
S	Soil factor
R_w	Response modification factor or ductility factor

The Z factor represents the design earthquake ground acceleration according to the zone in which the structure is located and has a 10 percent probability of exceedance in 50 years. This is nominally a 500 year event or an event with an annual probability of exceedance of 2×10^{-3}. This design load is modified by the other factors of the equation; performance drift limits are used. The importance factor is used to increase the design load for important structures; however, the 1.25 is not large enough to produce elastic response under a severe earthquake. Wen et al (1994) notes that "this small range (in I) is hard to justify since the uncertainty in the seismic excitation is generally so large that the different reliability levels required of the structure would lead to a much larger range of the structural resistance. To determine the importance factor rationally and qualitatively, a calibration of this value needs to be performed according to the performance goal required of the structure in terms of acceptable risks of limit states." The C factor is a function of the site soil conditions and the fundamental period of the structure.

The R_w factor allows for ductility in typical building structures and is also used for non-building elements. For non-buildings UBC Table 16-Q specifies R_w such as 3.0 for tanks. The ratio of C/R_w shall not be less than 0.5 The provisions call for

[1] The following sections contain various code provisions. The nomenclature and notation of the original reference was kept the same and is not necessarily consistent among references.

computation of the lateral force of the tank using the entire weight of the tank and its contents. A response spectra approach allowing for inertial effects of the contents is permitted.

Lateral force on elements and components shall be designed for:

$$F_p = Z\, I\, C_p\, W_p$$

where C_p is specified in Table 16-O for elements such as tanks, racks, anchorage, plumbing etc. and W_p is the weight of the element. Rigid elements are designed for 0.5 of their weight times the Z and I factors.

For equipment in facilities drift must be checked. Drift limits are specified in terms of the interstory displacement divided by story height, d, as:

$$d = 0.03/R_w \text{ and } < 0.004$$

Wen et al (1994) notes the drift is about 1.5 percent independent of the response modification factor. This is not consistent with a reliability based approach.

1997 Uniform Building Code – The 1997 Uniform Building Code is a transitional code going from the 1994 UBC to a national building code based on the NEHRP guidelines expected in the year 2000. Section 1632 present equations for calculating horizontal forces on nonstructural components and equipment.

1992 - 1995 NEHRP Provisions- The National Earthquake Hazard Reduction Program (NEHRP) has been used in waterfront design. The design earthquake is established as 10 percent chance of exceedance in 50 years which may result in both structural and non-structural damage which is expected to be repairable. For larger motions the intent is to preclude collapse. Peak ground acceleration maps are provided. The 1992 provisions computed seismic shear as:

$$V = C_s\ W$$

$$C_s = (1.2\, A_v\, S) / (R\, T^{2/3})$$

$$\text{but} \qquad C_s =< (2.5\, A_a) / R$$

where A_a and A_v are defined as effective peak acceleration and effective peak velocity-related acceleration. R is the response modification factor similar to the UBC but with different values. The 1994 provisions modified the equation by introducing amplification factors, F_a and F_v, and redefined the soil types into six groups:

$$C_s = (1.2\, A_v\, F_v) / (R\, T^{2/3})$$

$$\text{but} \qquad C_s =< (2.5\, A_a\, F_a) / R$$

The 1995 NEHRP soil site classes which establish values for F_a and F_v are defined as:

A) Hard rock with measured shear wave velocity, $v_S > 5{,}000$ ft/sec (1,500 m/s)

B) Rock with 2,500 ft/sec $< v_S < 5{,}000$ ft/sec (760 m/s $< v_S <$ 1500 m/s)

C) Very dense soil and soft rock with 1,200 ft/sec $< v_S <$ 2,500 ft/sec 360 m/s $< v_S$ 760 m/s) or with either $N > 50$ or $s_u \geq$ 2,000 psf (100 kPa) where N is average blow count SPT and s_u is average undrained shear strength

D)Stiff soil with 600 ft/sec $< v_S <$ 1,200 ft/sec (180 m/S $< v_S <$ 360 m/s) or with either $15 < N < 50$ or 1,000 psf $< s_u <$ 2,000 psf (50 kPa $< s_u <$ 100 kPa)

E) A soil profile with $v_S <$ 600 ft/sec (180 m/s) or any profile with more than 10 ft (3 m) of soft clay defined as soil with PI > 20, w > 40 percent, and $S_u <$ 500 psf (25 kPa)

F) Soils requiring site-specific evaluations:

1.Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays (H > 10 ft (3 m) of peat and/or highly organic clay where H = thickness of soil)

3.Very high plasticity clays (H > 25 ft (8 m) with PI > 75)

4.Very thick soft/medium stiff clays, H> 120 ft (36 m)

EXCEPTION: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

The 1995 NEHRP provides the following steps for classifying a site.

Step 1: Check for the four categories of Soil Profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), w > 40 percent, and PI > 20. If these criteria are satisfied, classify the site as Soil Profile Type E.

Step 3: Categorize the site using one of the following three methods with v_s, N, and s_u

a. v_s for the top 100 ft (30 m)

b. N for the top 100 ft (30 m)

c. N_{ch} for cohesionless soil layers (PI < 20) in the top 100 ft (30 m) and average s_u for cohesive soil layers (PI > 20) in the top 100 ft (30 m) where N_{ch} is average blowcount for cohesionless layers from SPT

The NEHRP provisions found in FEMA 223A Section 3.3.9 discusses storage tanks and allows either the AWWA or the API procedures. It specifies that pipe connections to steel storage tanks provide for 2 inches of vertical displacement for anchored tanks and 12 inches for unanchored tanks. It further specifies piping systems to be made of ductile materials; design strengths for service load combinations may be 90 percent of yield strength for ductile steel, aluminum, or copper, 70 percent of yield strength for threaded pipe made from ductile material and 25 percent of minimum tensile strength for plastic pipe. Threaded connections in piping constructed of nonductile materials shall not more than 20 of minimum specified tensile strength. Section 3.1.3 defines seismic force levels for tanks and piping.

1997 NEHRP/FEMA 273 - FEMA 273, 'NEHRP Guidelines for the Seismic Rehabilitation of Buildings" defines basic safety objectives by two earthquake levels, 10 percent probability of exceedance in 50 years and 2 percent probability of exceedance in 50 years. The first level treats life safety while the second addresses collapse prevention. Chapter 11 discusses nonstructural elements. The chapter presents two approaches for nonstructural rehabilitation. The first is a prescriptive procedure where published standards are used. The second procedure is an analytical procedure horizontal component forces are computed based on the spectra level, performance objective and component weight.

1997 NEHRP/FEMA 302 – FEMA 302 addresses design of new buildings and further develops the previous work. A set of maps are used to determine ground motion parameters, which are adjusted based on soil classes discussed above. The ground motion is taken as two-thirds of the 2500 year maximum considered earthquake. Response modification factors, R, system overstrength factor, Ω_o, and deflection amplification factor C_d are used. The basic form of the base shear equation is:

$$V = C_s W$$

where

$$Cs = S_{DS} / (R/I)$$

where

I The occupancy importance factor
R The response modification factor

The determination of horizontal forces on nonstructural elements and equipment is essentially the same as in FEMA 273.

DOE Criteria -The Department of Energy (DOE) guidelines define four classes of structure, General Use, Important / Low Hazard, Moderate Hazard, and High Hazard. The later two classes refer to nuclear facilities. This work establishes a risk acceptability criteria which has direct correlation to the containment of hazardous materials and lifelines. General Use facilities are typical ordinary structures to be designed by current code provisions. Important / Low Hazard facilities would include laboratories, computer centers, hazard recovery facilities and other facilities with a building code importance factor of 1.25. Moderate Hazard facilities include facilities where confinement of contents is necessary to protect personnel including the handling of radioactive and toxic materials. High Hazard facilities include facilities where confinement of contents is necessary for public and environmental protection such as nuclear facilities; these facilities represent hazards with potential long term and widespread effects. Specification of the design earthquake is established in terms of the annual probability of exceedance starting at a value similar to the UBC value and decreasing with class of structure. The annual probabilities of exceedance values expressed as earthquake nominal return times used for the four classes of structure are:

Structure Class	Earthquake Return Time (years)	Annual Exceedance Probability
General Use	500	2×10^{-3}
Low Hazard	1000	1×10^{-3}
Moderate Hazard	1000	1×10^{-3}
High Hazard	5000	2×10^{-4}

The DOE guidelines define the performance goals of each class of structure. Associated with each performance goal is a probability of the structural system meeting that goal. The following table shows that relationship.

Structure Class	Performance Goal	Probability of failure to meet goal
General Use	Occupant Safety Prevent major structural damage/collapse Code provisions	0.5
Low Hazard	Continued Operation Capacity to function, occupant safety, relatively minor structural damage	0.5
Moderate Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.1
High Hazard	Continued Functionality Hazard Confinement Limited damage to insure containment of hazardous materials	0.05

By combining the earthquake probability of occurrence and the probability of exceedance of failure shown above the annual probability of failure can be calculated and is shown below. These values range from 0.001 for general use structures in which the measure of performance is probability of collapse to 0.00001 for high hazard facilities in which the measure of performance is failure of containment of the high hazard. The goals and probabilities are:

Structure Class	Performance Goal	Annual Exceedance Probability of Failure
General Use	Occupant Safety	1×10^{-3}
Low Hazard	Continued Operation	5×10^{-4}
Moderate Hazard	Continued Functionality Hazard Confinement	1×10^{-4}
High Hazard	Continued Functionality Hazard Confinement	1×10^{-5}

This data is thought to be a significant statement on the general acceptability of seismic risk and as such has direct bearing on establishing guidance for comparable operations associated with oil terminal facilities.

Navy Criteria- The Navy uses NAVFAC P355 for the design of buildings and associated details. Provisions are included in this reference for design of lifeline supports

and equipment using 1990 SEAOC lateral force criteria. Chapter 11 presents a procedure for designing architectural elements. Chapter 12 addresses mechanical and electrical component anchorage while Chapter 13 deals with non-buildings and addresses tank design criteria. Chapter 14 gives an overview of utility systems and pipe details. The NAVFAC P355 (1992) does not reflect the most recent UBC and NEHRP provisions and is undergoing revision to conform to NEHRP.

The general lateral base shear applied to a structure is the product of the structure weight , a zone coefficient, an importance coefficient, and a site factor composed of soil type and structure period, all divided by a ductility factor based on the type of lateral force resisting system. The pseudostatic load is distributed along the height of the structure and resulting stresses and overturning moments determined. Combinations of dead load, live load and other loads are used and orthogonal horizontal loads are combined to produce a total. Drifts are checked. Allowable stress design is used with adjustment factors.

The general, code anchorage force to be applied to a structure for relatively small elements of equipment within a building is specified as the product of the equipment weight, a zone factor, an importance factor, and a factor describing the type of element. The element is limited to less than 10 percent of the total weight of the structure or to 20 percent of the floor weight at the element level. Drift limitations can apply. For self-supported equipment on the ground, the value of F_P may be reduced by 2/3. Large elements are designed as non-buildings using the provisions of Chapter 13. Pipes containing hazardous materials within a building require special provisions for flexibility such as, flexible couplings, expansion joints, and spreaders.

40 CFR 248 -USEPA 1991 The Resource Conservation and Recovery Act - This public law specifies the design requirements for municipal solid waste landfills (MSWLF). There is major concern that these dumps can pollute the ground water. The law states "new MSWLF units and lateral expansions shall not be located in seismic impact zones unless... all containment structures, including liners, lechate collection systems and surface control systems are designed to resist the maximum horizontal acceleration in lithified earth material for the site" The law mandates that a composite liner composed of a geomembrane and 2 feet of low permeability soil be used. The maximum acceleration is defined as emanating from a seismic event with a 90 percent chance of not being exceeded in 250 years; this is nominally a 2500 year return time event. Design criteria is given for allowable concentrations of toxic chemicals and acceptable values of hydraulic conductivity.

This legislation is a significant statement which establishes defined risk limits for seismic pollution of the environment and as such is applicable to comparable oil terminal facilities.

American Water Works Association D100, D103, D110 Standards. These standards describe the design of bolted and welded steel tanks and prestressed concrete tanks. Structures to be designed for Seismic Zones 1, 2 or 3 may be designed for a fixed percentage weight of 2.5 percent, 5 percent and 10 percent respectively. Elevated tanks are design using:

$$V = Z\,K\,C'\,S\,W$$

where

$$C = 1/(15\sqrt{T})$$

Specific values for K are given.

Tanks on the ground in Seismic Zone 4 require a pseudostatic design but allow for a response spectra. The horizontal base shear is given by:

$$V = Z\,I\,K\,S\{C_1\,(W_s + W_r + W_1) + C_2\,W_2\}$$

and the overturning moment is given by:

$$M = Z\,I\,K\,S\{C_1\,(W_s X_s + W_r H_t + W_1 X_1) + C_2\,W_2 X_2\}$$

Where

C_1	Factor based on natural period
C_2	Factor based on natural period
H_t	Total height of tank shell
I	Importance factor
K	Structure coefficient depends on type and anchorage
S	Soil factor
W_r	Weight of effective mass of tank roof
W_s	Weight of effective mass of tank wall
W_1	Weight of effective mass of tank contents moving with tank shell
W_2	Weight of effective mass of first mode tank sloshing
X_s	Height from bottom of tank shell to cg of shell
X_1	Height from bottom of tank shell to centroid of lateral force applied to W_1
X_2	Height from bottom of tank shell to centroid of lateral force applied to W_2
Z	Zone factor

The bolted steel tank standard uses an SC_1 value of 0.14. The fundamental period of the tank is prescribed by equation and varies depending on the particular standard and tank type. A fundamental period for the sloshing mode is also computed. The K value varies

with the type of tank and whether it is anchored or not. Unanchored tanks have higher K values.

Response spectra values can be substituted for equation values. The approach considers that the loading consists of components at the tank fundamental frequency and also components at the sloshing frequency. Response spectra values based on a tank period can be substituted for $ZIKSC_1$. Additionally, sloshing period values can be substituted for $ZIKSC_2$. Vertical force components can be included in the computation. The designer has the option to compute the resultant separately or in conjunction with horizontal forces. Tank wall stresses are computed from overturning moments and compared with allowable values. Formulas are given for computation of vertical compressive and tensile forces at the tank base. Flat-bottom tanks may be anchored or unanchored. Where tanks are unanchored the maximum thickened annular ring width at the base used to limit overturning is limited to 7 percent of the tank radius and the thickness shall not exceed the thickness of the shell thickness at the bottom. Anchored tanks could be susceptible to tearing if not properly designed. Hydrodynamic seismic tensile membrane forces are computed. Allowable stresses are increased by one-third for seismic forces. Guidelines are given for important foundation considerations including allowable bearing and the need for soil homogeneity across the foundation. Various types of tank foundations are discussed. The user shall specify the amount of tank freeboard for sloshing. Failure to provide for sloshing will damage the roof if the tank is completely full. Provisions are included to allow for local site conditions. A 2 percent damped curve is recommended for design of the structure and a 0.5 percent damped curve is recommended for sloshing of the liquid. The amplified acceleration shall be determined for the cantilever beam period of the shell and effective portion of the contained fluid. When site response return times are not given a maximum credible event or 10,000 year return time event can be used with a response reduction factor not to exceed 2.6.

The AWWA has standards for ductile iron, steel, concrete, and asbestos pipe; however they do not address seismic design directly.

American Petroleum Institute Standard 650- The American Petroleum Institute provisions follow 1980's code design and was revised and updated as recently as 1996. The tank overturning moment is:

$$M = Z\,I\,\{C_1\,(W_s X_s + W_r H_t + W_1 X_1) + C_2\,W_2 X_2\}$$

where the terms are the same as defined for the AWWA equation above. The term C_1 is set at 0.60 unless the product of $Z\,I\,C_1$ and $Z\,I\,C_2$ are determined from response spectra. The term C_2 is defined by:

$$C_2 = 0.75\ S/T \qquad \text{for } T <= 4.5$$

$$C_2 = 3.375\ S/\ T^2 \qquad \text{for } T > 4.5$$

If a spectrum is used for the factor Z I C_1, it should be developed for a damping coefficient of 2 percent of critical. The spectrum for the factor Z I C_2 should be based on the spectrum for Z I C_1 but with a damping coefficient of 0.5 percent of critical.

> Summers (1997) reports that extensive experimental studies and observations during past earthquakes have demonstrated that the radial length of uplifted bottom plate, and hence, the actual liquid weight resistance which is mobilized during an earthquake is underestimated by the API uplift model. He explains the reasons for this are that the API model does not account either for the in-plane stress in the bottom plate, or for the dynamic nature of the tank response. The API model also calculates a somewhat narrow compression zone at the toe of the tank, thus leading to large compressive stresses in the tank shell for relatively low overturning moments. Finally, the API approach does not account for the effect of foundation flexibility on the tank wall axial membrane stress distribution. These factors err on the conservative side and result in overdesign. The API procedure is recognized as a conservative approach and is acceptable for new tank design.

40 CFR 112: 38FR 34164 Environmental Protection Agency Regulations On Oil Pollution Prevention This public law applies to oil storage or processing facilities which are potential pollution sources. It does not apply to facilities where the storage capacity is 1,320 gallons or less and no single tank has a capacity in excess of 660 gallons. For facilities falling under provisions of this law, appropriate secondary containment is mandated such as dikes, curbs, sumps or ponds.

State of California Above Ground Storage Act of 1991 This law applies to sites containing petroleum/hazardous material storage tanks where the above ground storage capacity is over 1,320 gallons or where a single tank exceeds a capacity of 660 gallons. The law requires inspections, licensing and monitoring. The foundation system must be designed to allow for early detection of releases of materials before reaching the ground water.

American Railway Engineering Association, Chapter 9 Seismic Design For Railway Structures The procedure specifies three levels of ground motion: A Level I ground motion has a reasonable probability of being exceeded during the life of the structure and the structure is at a serviceable limit state which requires it to remain elastic. Only moderate damage which does not affect trains at restricted speeds is allowed. Allowable stresses are increased 150 percent in steel and 133 percent in concrete elements. The return period for a Level I earthquake is between 50 and 100 years. The determination of a specific ground motion level is left to the designer based on the type and volume of traffic expected. A Level II ground motion has a low probability of being exceeded during the life of the structure and represents a limit state to ensure overall structural integrity. The structure may respond in the inelastic range but ductilities are limited. The return period for a Level II earthquake is between 200 and 500 years. The selection of the

specific level is left to the designer based on overall economics considering structure cost and train schedules. A Level III ground motion is established for a rare intense earthquake which establishes a survivability limit state which allows extensive damage but precludes collapse. Foundation failures are limited so as not to cause major changes in the structure geometry. The return period for a Level III earthquake is between 1000 and 2500 years. The selection of a specific level is left to the designer based the consequences of loss of the structure and include costs of construction, loss of use, existence of alternate routes and location of the bridge. Pseudostatic, spectral and dynamic procedures are used depending on the type and irregularity of the structure. The nominal 100 year, 500 year and 2500 year return time peak horizontal rock accelerations are specified on a national map

Standard Specification For Highway Bridges, AASHTO This is a national code and as such divides the US into regions based on levels of expected ground motion. A map is provided which shows peak horizontal rock accelerations with a 90 percent probability of not being exceeded in 50 years which is a nominal 500 year return time event. Two categories of bridge structure are defined, essential bridges which are expected to function after a design earthquake and other bridges which are designed for near elastic response at moderate events and for limited damage at the maximum credible event. Four categories A through D are defined to treat importance and variation in seismic acceleration potential. A and B are low treat level requirements while D is highest representing an essential structure in the highest exposure zone. Three site profiles are defined and serve to define site amplification. Elastic earthquake lateral forces are determined based on the map accelerations and site soil factor. Component response modification factors are used to reduce the elastic forces for substructure elements while connections of superstructure to abutment and expansion joints are increased. The modification factors are analogous to ductility factors. It is assumed that columns will yield when subjected to forces from the design ground motion but that the connection will be able to resist the deformations with little damage. Wall piers have minimal ductility and an R value of 2 was assigned. Well designed columns in a multi-column bent have good ductility and a value of 5 was assigned to them. Single columns lack redundancy thus a value of 3 was assigned. For C and D bridges the connections are designed for the maximum forces that can be developed by plastic hinging in the columns. The probability of elastic force levels not being exceeded in 50 years is in the range of 80 to 90 percent. Procedures are given to calculate displacements. Modal response techniques are used in the analysis of response. It is suggested that a factor of safety against liquefaction be 1.5 for important bridges. Guidance is given for pile design

1990 CALTRANS CALTRANS criteria was developed for non-buildings and is of general interest. It is summarized as:

$$V = ARS\ \ W / Z$$

where W is the total weight and Z is an adjustment factor for ductility and risk and based on the period and type of structural element. ARS is the 5 percent elastic response

spectrum at the site in g's based on the maximum expected acceleration at bedrock or rocklike material. The seismic force in two directions is required and to be evaluated by adding 30 percent of the force to the component in the perpendicular direction. For conservative design, the vector sum can be used. A load factor of 1.0 is used and live load is not included. The strength reduction factor, ϕ, for concrete columns can be increase from 0.9 to 1.2 to recognize an increase in strength from well confined concrete.

Japan Gas Association Recommended Practice For Pipelines- The 1978 Miyagiken-oki earthquake caused heavy damage to the gas distribution system in Sendai City. Damage was concentrated in threaded steel pipelines of about 2-inch diameter. As a result of this guidelines were developed for Japan. Japan is divided into four seismic zones and three soil classifications are used. The seismically induced horizontal ground deformation is estimated by:

$$U = \alpha_1 \alpha_2 U_0$$

where

α_1 Constant based on site location in the range of 0.4 to 1.0
α_2 Constant based on soil condition and importance in the range of 0.5 to 1.8
U_0 Constant which is set at 5 centimeters

The vertical displacement is half of the horizontal. The guide outlines four load deformation conditions shown in Figure 5-1 and a Deformability Index is used to estimate pipe capacity. The Deformability Index includes strain capacity of the pipe and of the joint.

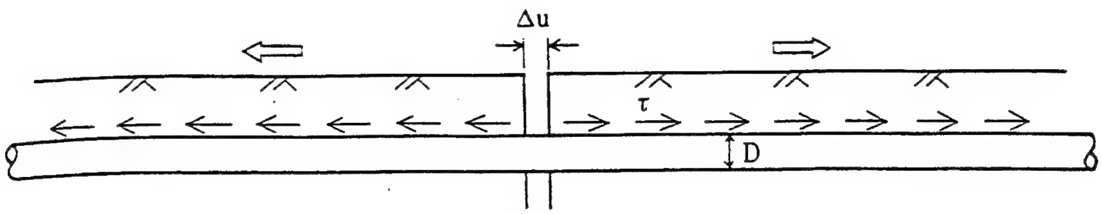
IEEE Standard 344-1987- The IEEE has developed a standard for the seismic qualification of equipment for the nuclear industry.

Performance Objectives

The development of performance objectives is the first step in development of a general criteria for lifelines. The following performance objectives are presented herein and represent a new synthesis proposed for use. They are based on mandates of public law and extensions of current criteria.

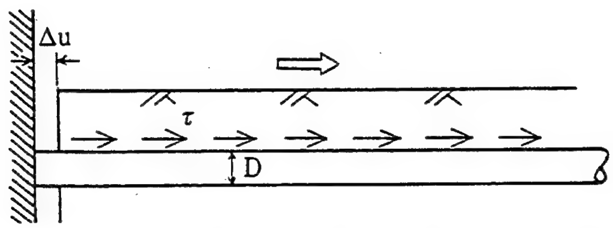
Ordinary Construction / Ordinary Lifelines - Lifeline service associated with construction categorized as "ordinary" shall be designed with the same levels of service. In general ordinary construction is expected to

- Resist a moderate level of ground motion without damage;

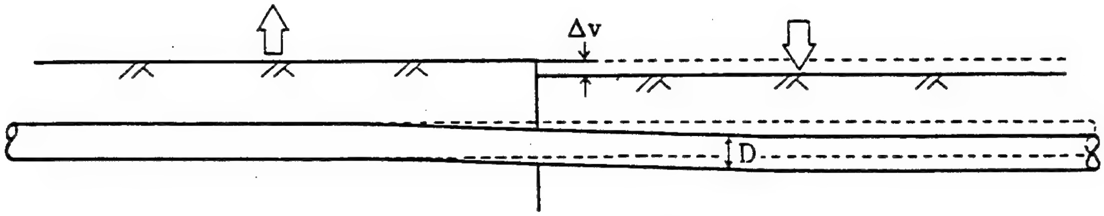


Δu : Allowable ground displacement = Deformability Index (Horizontal)
τ : Soil Restriction

a. Horizontal Relative Displacement between two Ground Blocks

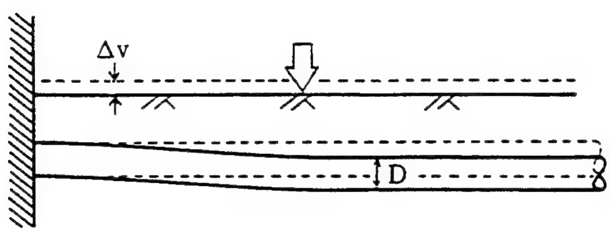


b. Horizontal Displacement in a Ground Block Adjacent to a Rigid Structure



Δv : Allowable ground displacement = Deformability Index (Vertical)

c. Vertical Relative Displacement between two Ground Blocks



d. Vertical Displacement in a Ground Block Adjacent to a Rigid Structure

Ground-Displacement Models for Deformabilitty Evaluation for Buried Pipelines

Figure 5-1. Ground-displacement models for pipe deformation.

- Resist a major level of earthquake ground motion without collapse, but with structural as well as nonstructural damage.

Wharves and Piers Lifelines associated with pier or wharves shall be designed with the same levels of service.

- Resist a moderate level of ground motion without damage;
- Resist a major level of earthquake ground motion without collapse, and with the structural in a repairable condition.

Essential Construction / Essential Lifelines - Lifeline service associated with construction categorized as "essential" shall be designed with the same levels of service. In general essential construction is expected to:

- Resist the earthquake likely to occur one or more times during the life of the structure with minor damage without loss of function and the structural system to remain essentially linear.
- Resist the rare earthquake with a low probability of being exceeded during the life of the structure without failure and without loss of acceptable levels of functionality.

Hazardous Materials/Lifelines - Lifeline service associated with construction categorized as "containing hazardous materials" shall be designed with the same levels of service. In general hazardous material containment construction is expected to:

- Resist pollution and release of a major spill of hazardous materials for a very rare event

Seismic Loads

The second element of a general criteria for lifelines is the specification of seismic load level to establish the ground motion and lateral load forces to be applied in design. It is based on current criteria and an extension of existing mandates logically applied to analogous situations.

Design Earthquakes

The following criteria are based on current criteria and public law. The lifeline systems shall be designed to resist the loading produced as follows:

- **Ordinary category of construction on average seismicity sites**
  For sites of average seismicity, use NEHRP provisions, which establishes the earthquake at a nominal 10 percent chance of exceedance in 50 years.

- **Pier or wharf category of construction**
  Sites where the lifeline is associated with a pier or wharf shall use a two-earthquake procedure with Level 1 and a Level 2 based on a local site seismicity study. Values less than NEHRP code are not be permitted

- **Essential category of construction**
  Sites where the lifeline is deemed important and essential shall use a two-earthquake procedure with Level 1 and a Level 3 earthquake based on a local site seismicity study. Values less than code are not be permitted.

- **Construction containing polluting or hazardous material**
  A Level 4 earthquake exposure shall be used.

In addition to seismic ground motion there are additional hazards which must be considered:

- Fault movement and ground displacement
- Liquefaction and associated lateral spreading, settlement flow slides, loss of support and buoyancy of buried tanks.
- Landslides
- Tsunamis

Modification to Design Ground Motion

Lifelines consist of a variety of elements some of which are substantial structures such as tanks, transformer stations and bridges, others are distributed elements such as buried pipelines, power lines and railroad tracks, and others are components within structures such as internal equipment, transformers, and other building elements. The ground motions used in design of lifelines may differ from the motions used in conventional building design since the seismic motion on the lifeline may be substantially different than that associated with free-field ground motion. For component elements located within a structure the lifeline component design motion can be substantially amplified by the response of the structure. In such cases the motion to be used for design of the component must be the local seismic motion transmitted by the structure to the component. The dynamic coupling between the component and the structure must be taken into account if the component is of a size sufficient to influence the response of the structure. Large differential motions may be produced on components which are supported at multiple locations.

Chapter 6 of NAVFAC P355.1 illustrates the procedure for calculation of the maximum floor accelerations using the linear response spectra technique. A modal participation factor is applied to the story modal acceleration response to determine the

modal spectral acceleration to be applied to the lifeline component. A design response spectrum is constructed using the modal floor accelerations, the participation factors, a magnification factor, and the period of the lifeline component. Response spectra techniques have been utilized for at least the last 35 years. They offered a means for performing dynamic analysis more accurately than pseudostatic approaches. The response spectra technique is a linear procedure. A structure responding to a major earthquake is expected to sustain significant nonlinear behavior. The ability of response spectra techniques to accurately track displacements reduces as the amount of nonlinearity increases. With the evolution of the desktop computer, nonlinear finite element techniques which previously required extensive mainframe computer time, have now been developed which can offer a potentially more accurate analytical alternative.

Lifeline Performance During Recent Earthquakes

Understanding the behavior and possible failure mechanisms of a lifeline structure is important in the development of a design criteria for safe operation. Part of understanding the performance of a lifeline structure in an earthquakes involves understanding the design from which the structure was constructed. and the construction practice used in its erection. Werner and Hung (1982) gives an excellent compilation of case studies mostly recounting Japanese experiences from the 1920's to 1980. They conclude that "By far the most significant source of earthquake-induced damage to port and harbor facilities has been porewater pressure buildup... which has led to excessive lateral pressures applied to quay walls and bulkheads." They cite the 1964 Niiagata and 1964 Alaska earthquake where "porewater pressures buildup has resulted in complete destruction of entire port and harbor areas" They note that direct effects of earthquake induced vibrations on waterfront structures is minimal and overshadowed by liquefaction induced damage. In the 1978 Sendai earthquake a major oil refinery with 90 storage tanks had three fail and three damaged. Additionally a large welded steel plate tank pulled out of its concrete embedment. A summary of recent lifeline experiences during earthquakes follows.

Alaska Earthquake - The 1964 caused considerable damage to oil storage tanks by tsunamis, earth settlement, and liquefaction. Damage to Union Oil tanks in Whittier caused fires. In Anchorage seven tanks collapsed releasing combustible fluids; three additional Standard Oil tanks released 750,000 gallons of aviation fuel. This experience led to a change in tank design, Eguchi (1987)

San Fernando Earthquake- The 1971 San Fernando earthquake resulted in direct losses to the electric power systems of $33 million. It caused distress to numerous tanks. Bulging of the lower section of about 12-inches above the base was noted extensively and termed "Elephant's Footing". Ductile steel pipelines were able to withstand ground shaking but could not withstand ground deformation associated with faulting and lateral spread. Eleven transmission pipelines were damaged by liquefaction induced lateral spread and landslides. Eighty breaks occurred to the underground welded steel

transmission pipeline located in the upper San Fernando Valley, the most serious in a 1930 old oxyacetylene-welded pipeline. Although located in an uplift zone the failure was caused by compressive forces wrinkling the pipe, Eguchi (1987). Newer pipelines in the same area did not fail. There were 18 documented hazardous material releases resulting in 6 fires. There was damage to the Jensen water treatment plant resulting in an outage. The absence of an inlet to outlet bypass was noted as a factor in impeding the problem of restoration of service.

Santa Barbara - In the 1978 magnitude 5.1 Santa Barbara earthquake a train derailed shortly after the earthquake from damaged tracks. About 40 cars derailed at a speed of 50 miles per hour.

Coalinga Earthquake - The 1983 Coalinga earthquake had adequately designed pipelines which remained serviceable; however large vertical tanks containing molasses tilted and in one case overturned. This was initiated by large deformations in the steel support frame. At a treatment plant, chlorine tanks on standard saddle supports slid up to 10 inches. The valves on a 1-inch line to a clarifying tank shook open causing a major oil spill. Anchors on a 12 kV transformer broke. A hazardous material spill resulted in significant damage to a high school; there were three other hazardous material incidents of significance. Numerous breaks in the natural gas line occurred but fires did not occur since the main valve was closed manually shortly after the earthquake. Several tank and pipeline failures occurred in oil drilling and processing facilities. In general it was noted that secondary containment systems functioned well. most pipe breaks occurred at pipe connections.

Whittier Narrows- The 1987 magnitude 6.1 earthquake demonstrated that well designed process pipelines can perform well. Damage where it occurred was usually limited to sections that were corroded or anchored at two locations which experienced large lateral relative displacement. A 1-ton relocatable gas cylinder being filled with chlorine started to roll down the loading platform breaking the connection causing a significant chlorine release. Southern California Gas reported 1411 gas leaks were directly caused by the earthquake. Portions of the California State University, Los Angeles were without gas for 12 weeks. Five fires were reported; three of these were attributed to gas leaks. There were about 30 hazmat calls for assistance.

Loma Prieta Earthquake- The 1989 magnitude 7.1 Loma Prieta Earthquake caused failure of many pipelines and tanks. The Port of Redwood City is located at the southern end of the San Francisco Bay. The Port contains tanks for petroleum. The Port was constructed on Bay Mud. Damage consisted primarily of broken water lines and damaged batter piles. The Port of Richmond is located at the northeast end of the San Francisco Bay and handles petroleum products and liquid bulk cargo. Portions of this port are constructed on rock and other portions on fill. The primary damage was the rupture of a gasoline storage tank at the UNOCAL terminal. Fuel was contained in the surrounding berm. Some liquefaction was reported in undeveloped areas of the port. Broken waterlines occurred at the Ford plant from liquefaction and excessive soil pressures. The

Port of San Francisco is located on the west side of the San Francisco Bay and handles general cargo. The port is constructed on fill. The primary damage was liquefaction and settlement. Numerous buildings were damaged water and gas lines broke. The Port of Oakland is located on the east side of the San Francisco Bay on fill. The port sustained wharf damage and noted batter pile failures. Liquefaction of the fill produced settlement and lateral spread. Horizontal accelerations were measured at the wharf and ranged from about 0.3g to 0.45g. Cranes suffered damage and water lines broke. Fire lines ruptured eliminating fire fighting protection, Seed et al. (1990). Tank failure modes consisted of "elephant's foot " bulging, vertical splitting of tank wall, puncture of the tank wall by restrained pipe, pipe damage from differential anchorage motion. Hazardous material spills occurred in several industrial and a few commercial facilities. Over 300 liquid hazardous spills occurred in the San Francisco and Monterey Bay areas as a result of ruptured tanks, pipe leaks, equipment leaks, and broken containers. It appears that secondary containment was generally effective. At least 50 instances occurred of release of hazardous gases other than natural gas. There were 3 to 4 leaks on a high pressure gas main and between 300 to 400 leaks on low pressure gas lines.

The Navy sustained 44 pipeline breaks in pipes up to 16 inches in diameter on Treasure Island. They included 28 fire and freshwater lines of steel or asbestos cement, 10 sewage lines of vitrified clay and 6 welded-steel gas lines, Egan and Wang (1991). Many of the breaks occurred near the dike in areas of high lateral spreading. Crude estimates of lateral spreading required to cause failure are:

Type	Pipe Diameter	Spreading to Induce Failure
Steel or Asbestos Cement	1 to 4 in	1 inch
Steel or Asbestos Cement	12 to 16 in	6 to 12 inches
Vitrified clay pipe		1/4 inch

Soil liquefaction caused damage to the terminal facilities much of which were on filled land composed of loose dumped or hydraulically placed sand underlain by soft normally consolidated Bay Mud. Liquefaction of the fill resulted in settlements and lateral spreading, cracking the pavement over a wide area. Maximum settlements of the paved yard area were up to a 12 inches.

In the Monterey area water tanks belonging to PG&E were damaged and one ruptured apparently as a result of foundation softening and displacements. Settlements of several inches were noted and there were breaks in utility lines. Pile supported facilities were not damaged.

Transportation facilities sustained $1 billion in damage including $200 million to the Cypress Street elevated viaduct. Numerous roads were closed by pavement damage, landslides, or bridge damage. A 3000-foot section of runway was severely damaged having several breaks as large as 30 inches in width. Undulations were noted in the pavement along with settlement. The pavement was situated was on 10 to 15 feet of unconsolidated hydraulically dredged sand fill which experienced extensive liquefaction. The runway at the Naval Station, Alameda cracked and moved laterally from liquefaction of the soil below. The Port of Oakland experienced liquefaction damage to paved yard areas Batter piles in wharves were damaged.

Big Bear Earthquakes- On June 28, 1992 two earthquakes occurred in San Bernadino County, California, a magnitude 7.5 at 4:58 AM and a magnitude 6.6 at 8:04 AM. These two events were followed by numerous aftershocks. Horizontal fault rupture displacement associated with these event was from 5 to 9.5 feet. Most pipeline damage was associated with the rupture zone. At least 6 water tanks ranging in size from 42,000 to 417,000 gallons were damaged. Damage consisted of elephant's foot bulging at the base, shell and roof damage, shell splitting at access hatches and broken pipe entering the tanks.

Guam Earthquake - On August 8, 1993 a magnitude 8.1 earthquake occurred 50 miles offshore and caused over $125 million in damages to Naval facilities on Guam. Nearly all of Guam is firm soil or rock except for the region containing the commercial and Navy ports which is composed of natural alluvium and artificial fill. It is estimated the peak horizontal ground accelerations were about 0.25g. Liquefaction was a major problem and lateral spreading of 1 to 2 feet was observed at wharf areas. It also resulted in settlements, backfill collapse and bulkhead movements. Buried water and power lines were fractured. Sheet piles failed in shear and deadman anchors pulled out. Pier batter piles failed in shear at the pile cap. Other Navy damage consisted of fuel tank leaks, sloughing of a dam, damage to masonry housing units and major damage to the power plant which supplied 20 percent of the islands power capacity.

Northridge Earthquake- On January 17 1994 a moment magnitude 6.7 earthquake occurred in Northridge. This event caused about 1,400 water, gas and fuel pipeline breaks in the San Fernando Valley area. Many of the breaks occurred in mapped areas of high liquefaction potential. Outside the zone of high liquefaction potential, the dispersed pattern of breaks is attributed to old brittle pipes damaged by ground movement. While much of the pipe damage is within the liquefaction zone, this did not correlate to areas of high structural damage in that a large amount of structural damage occurred outside the zone of high liquefaction potential. In the Granada Hills area pipe breaks from water mains resulted in soil erosion and formation of large craters. On Balboa Boulevard a 22-inch pipe suffered two breaks, one in tensile failure and the other in compressive failure. These pipe failures were located in a ground rupture zone perpendicular to the pipeline. Leaking gas ignited at several locations. Some broken water and gas lines were found to have experienced 6 to 12 inches of separation in extension. The area experienced widespread ground cracking and differential settlements. Liquefaction was not evident

on the surface and may have occurred at depth leading to subsurface soil block movement. Some of the surface cracking was associated with underlying bedrock movements associated with primary or secondary faulting. A 85 inch sewage pipe ruptured in the Jensen Filtration Plant and a large reservoir settled 2 to 4 inches. The San Fernando Power Plant Tailrace, a 600 by 110 foot asphalt lined pond was breached. Lateral spreading was noted. A water storage tank east of Highway 5 at Valencia Boulevard collapsed. The Port of Los Angeles sustained peak horizontal accelerations on the order of 0.1 to 0.2g which resulted in liquefaction of hydraulic fill damaging crane rails, disruption of utilities, ground cracking and lateral spreading of up to 6 inches. All of the damage was of a relatively minor nature.

Kobe Earthquake- On January 17 1995, the Hyogo-ken Nambu (Great Hanshin Kobe) earthquake, Japanese magnitude 7.2 (about 6.9 moment magnitude), occurred in Kobe Japan. This event produced major damage to Japan's second busiest port, Matso (1995). Liquefaction was a major contributor to the extent of the damage producing typical subsidence of a half meter. Piles were used extensively in this area. They were designed to account for the negative skin friction and additional ground improvement was also performed. Structures on such piles performed well even though major subsidence occurred in surrounding areas. Other structures not on piles suffered differential settlement and tilting and significant damage. Liquefaction caused up to 3 meters of lateral spread displacement, sunk quay walls, broke utility lines, and shut down 179 out of 186 berths at the port. Numerous tank failures were reported, mostly caused by uplift of unanchored tanks. One LNG tank cracked requiring the evacuation of 80,000 people. Six well-braced large spherical tanks sustained no damage. Liquefaction was responsible for major damage to crane foundations. Hydraulic fill behind concrete caisson perimeter walls fill liquefied causing the caissons to move outward, rotating up to 3 degrees, and settling from 0.7 to 3.0 meters. The caissons were designed for a lateral coefficient of 0.1g. A seismic coefficient of 0.2g was usually used in the design of dockside cranes. Peak accelerations of 0.8g in the NS direction, 0.6g in the EW direction and 0.3g vertical were noted from accelerograph recordings. The event had a duration of about 20 seconds. Most damage is attributed to liquefaction of backfill and associated pressures and settlements and lateral deformations since structures supported on piles suffered much less damage, Liftech (1995). It should be noted that caissons designed for 0.25g sustained lower levels of damage.

Liquefaction And Lifelines

Design of structures shall include provisions to evaluate and resist liquefaction of the foundation and account for expected potential settlements and lateral spread deformation. Special care will be given to buried pipelines in areas subject to liquefaction to preclude breaks resulting in release of a major spill of hazardous materials. The most important element in seismic design of pipelines is proper siting. It is imperative to avoid areas of landslide and lateral spread.

The presence of any potentially liquefiable materials in foundation or backfill areas shall be fully analyzed and expected settlements computed. Specific attention shall be paid to the acceptability of the amount of settlements. Since liquefaction is a major damage mechanism at the waterfront, remediation is a mandatory requirement where the risk of a pipeline break or tank failure is shown by computation to be possible and hazardous materials would be expected to be released.

Water, Gas And Liquid Fuel Lifelines

Pipelines

Pipelines must be designed to resist the expected earthquake induced deformation state and induced stresses. It is common practice to design the pipe support or embedment based on the nature of the soil encountered. Where marginal soil is encountered, pipelines can be supported on piles above ground or placed within larger pipes to allow ground movement. Generally permissible tensile strains are on the order of 1 to 2 percent for modern steel pipe. This is based on observation that steel pipelines have been observed not to rupture at tensile strains between 2 to 5 percent. Higher local strains have been noted. Pipelines have experienced wrinkling in compression at strains much less than the tensile limits; however this does not of itself constitute failure. A rule of thumb states that the onset of wrinkling occurs at strains of about 0.3 times the ratio of wall thickness to radius. Welded steel pipes have performed well during earthquakes. The quality of weld is very important. There appears to be more failures with oxyacetylene-welded steel pipes compared to arc-welded steel pipes. The difference may not be the type of weld but may be the weld quality. Corrosion of pipelines reduces their ability to withstand seismic forces. Pipeline damage seems inversely proportional to pipe diameter caused by an increase in stiffness with larger size pipe which makes it more able to resist deformation. Expressed in another form, pipeline strength is proportional to diameter. An exception to this seems to be steel pipe with a lap welded joint where strength decreased with increasing diameter. Also gasketed joints seem to be 5 or more times more likely to fail than welded joints. In addition to tensile and compressive failures, buckling failures are possible. The presence of bends, elbows and local eccentricities tends to concentrate deformation at these locations.

To accommodate differential motion between pipelines and storage tanks it is recommended that a length of pipeline greater than 15 pipe diameters extend radially from the tank before allowing bends and anchorage and that subsequent segments be of length not less than 15 diameters. Flexible couplings should be used on long pipelines. In general pipes should not be fastened to differentially moving components; rather, a pipe should move with the support structure without additional stress. Unbraced systems are subject to unpredictable sway whose amplitude is based on the system fundamental frequency, damping and amplitude of excitation. For piping internal to a structure, bracing should be used for system components. Flexible grooved pipe couplings can

reduce the transmission of stresses and resilient gaskets can dampen vibration. Manufacturers specification give guidance on linear and angular movement tolerances. "Grooved-end mechanical pipe couplings do not simultaneously provide maximum linear and angular movement. However, systems designed with enough joints, thus allowing for recommended tolerances, will accommodate both", Greene (1993). When large movements are anticipated seismic swing joints composed of flexible couplings, elbows and nipples can be used. Provisions for expansion must be included.

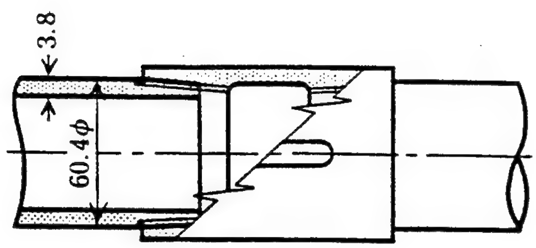
Machinery and pumps are often acoustically isolated by use of loose connections to minimize vibration transmission such as by use of slotted holes. Snubbers by definition are restraints with an air gap. Such anchorages can amplify seismic motion by having equipment bang against restraints. Use of resilient grommets or molded epoxy grouting can eliminate the air gap and avoid hard surface contact. The snubber and the connection of the snubber to the equipment and structure must have sufficient strength to transmit the inertial forces. The Northridge earthquake has shown that use of rails is not a satisfactory method of restraint and such usage was associated with many failures of welds and dislocation from the rails. Suspended pipelines can also resonate with the earthquake if not sufficiently restrained. Sway of suspended components must be restrained. Seismic isolation can be an effective technique for reducing loading on floor mounted equipment. Seismic isolation can be used in addition to snubbers or can be made a part of the snubber. While there are no standards for seismic snubbers, their capacities should be stated by the manufacturer and a rating is assigned by The Office of Statewide Health Planning and Development of the State of California. Proper anchorage capacity including both horizontal shear and overturning uplift is required and a wedge anchor is recommended. Poured in place anchors are not feasible for snubber tie-down since equipment location is variable and often not defined specifically. Snubbers must be omnidirectional with at least a 3/8 inch resilient collar at least 4 snubbers must be used and all snubbers must be rated, Lama (1994).

Nishio (1992) presents information of pipeline design in Japan. Figures 5-2 and 5-3 show pipe joint capacities for several Japanese pipe couplings. This excellent reference illustrates how the Japanese Gas Association provisions were developed and provides example calculations of their Deformability Index. The paper presents a discussion of the provisions and notes that the provisions use a value of deformation of 5.0 cm independent the liquefaction potential. Nishio introduces a probabilistic basis for assessing damage based on sample size. He shows the deformation capacity increases with diameter of the pipe.

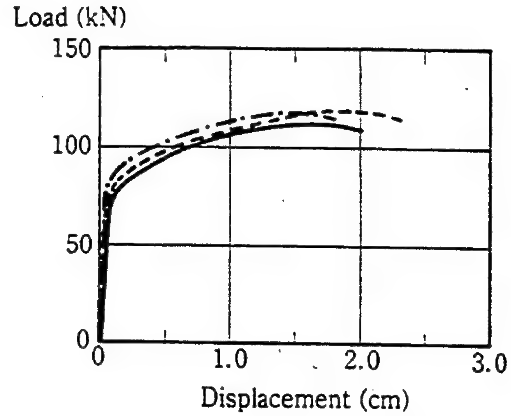
In the analysis of continuous pipelines, it is possible to estimate the axial strain of the pipe in terms of the maximum ground strain:

$$\varepsilon_{p,\max} = V_{\max} / c_p$$

and the maximum curvature of the pipeline

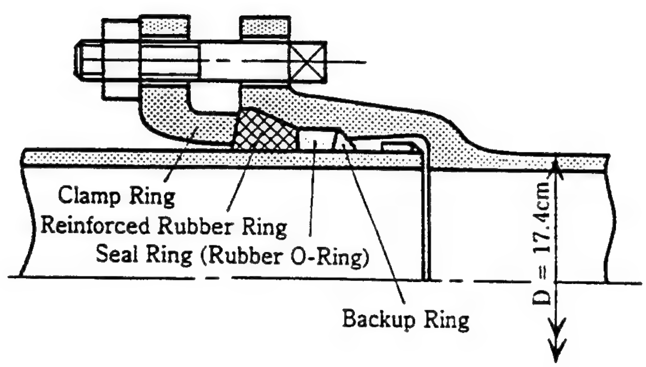


a. Cross Section of Thread-Joint (Socket, 50 mm)

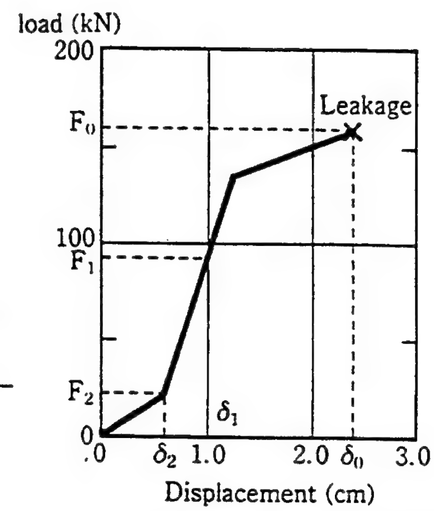


b. Load-Displacement Diagram of 50 mm Thread-Joint (Axial Tension)

Cross Section and Load-Displacement Diagram of 50 mm Thread Joint



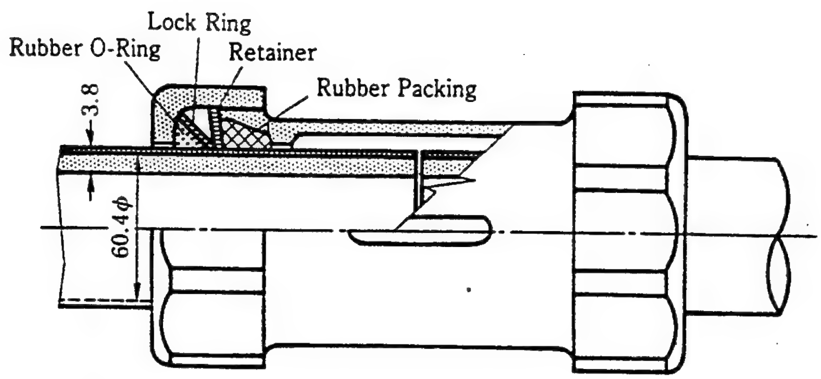
a. Cross Section of TM Joint



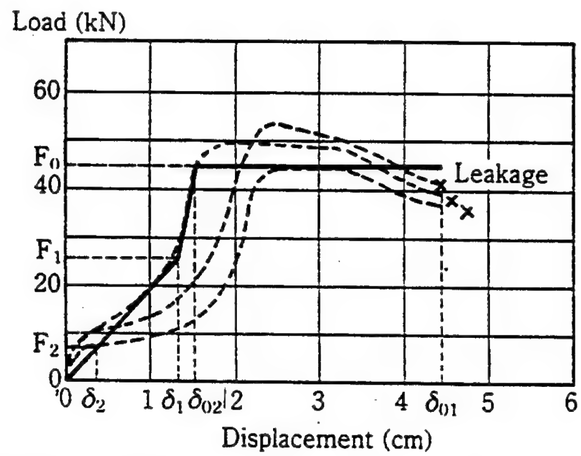
b. Load-Displacement Diagram for 150 mm TM Joint

Cross Section and Load-Displacement Diagram of Tokyo-Gas Type Mechanical Joint (TM Joint) for Ductile Cast-Iron Pipe (150 mm)

Figure 5-2. Cross section of Japanese pipe joints and strength.



a. Cross Section of Mechanical Joint for Steel Pipe (Tokyo Gas LM-Type Socket, 50 mm)



b. Load-Displacement Diagram of 50 mm Mechanical Socket
----- Experiments
——— Diagram for Calculation

Cross Section and Load-Displacement Diagram of Mechanical Joint for Steel Pipe (50 mm Socket)

Figure 5-3. Cross section of Japanese pipe joint and strength.

$$\chi_{p,max} = A_{max} / c_s^2$$

where

A_{max}	Maximum ground acceleration
C_p	Compression wave propagation velocity
C_s	Shear wave propagation velocity
V_{max}	Maximum ground velocity

The maximum pipe joint displacement and joint rotation can be estimated by:

$$U_p = \varepsilon_{p,\,max} L$$

$$\theta_p = \chi_{p,max} L$$

where

L Length of pipe segment

Note that C_s and C_p can be estimated from G and ρ as follows:

$$C_s = \sqrt{\frac{G}{\rho}} \quad \text{and} \quad C_p = \sqrt{3C_s}$$

Eguchi et al. (1994) present an analysis of lifeline system damage which gives a good insight into the performance of pipelines. Table 5-1 presents relative performance of various types of pipe to shaking, liquefaction, landslide and fault rupture. They have compiled data on the number of repairs per 1000 feet of pipe and developed Figure 5-4 for fault rupture and ground shaking. The symbols are identified in Table 5-1. They note that the two mechanisms of ground displacement/fault rupture and shaking are different with the former being more damaging. Figure 5-5 shows their estimate of relative pipe performance under liquefaction and landslides conditions. Pipe diameter while a factor in pipe performance it was found that pipe material and joint type were more significant factors in normalizing field data. The data is intended to give system relative performance and not to be used to evaluate a single pipe Wang et al. (1992) illustrate use of flexible pipe joints, Figure 5-6.

The provisions of NAVFAC P355 Chapter 12 Section 12-7d pertain to design of essential pipelines and are part of this specification. They are as follows:

d. Seismic restraint provisions. Seismic restraints that are required for piping ..... will be designed in accordance with the following provisions.

Table 5-1

RELATIVE PIPELINE VULNERABILITIES

Pipe Type	Ground Shaking (MMI)		Liquefaction, Lurching	Landsliding	Fault Rupture Displacement, (Inches)	
	8	10			10	100
WSAWJ (Modern)	L	L	L	L	L	L
WSAWJ (Pre-WWII)	L	L	M	M	M	M
WSGWJ (Pre-WWII)	M	H	H	H	H	H
WSCJ	M	M	H	H	H	H
CI	M	M	H	H	H	H
AC	M	M	H	H	H	H

Source: Eguchi (1983)

Pipe Types:

WSAWJ	=	Welded Steel Pipe with Arc-Welded Joints
WSGWJ	=	Welded Steel Pipe with Gas-Welded Joints
WSCJ	=	Welded Steel Pipe with Caulked Joints
CI	=	Cast Iron
AC	=	Asbestos Cement

Vulnerability Levels:

Low	=	No pipeline leaks expected.
Moderate	=	Some repairs expected.
High	=	Likely to fail or require repair given this level of seismic intensity or effect.

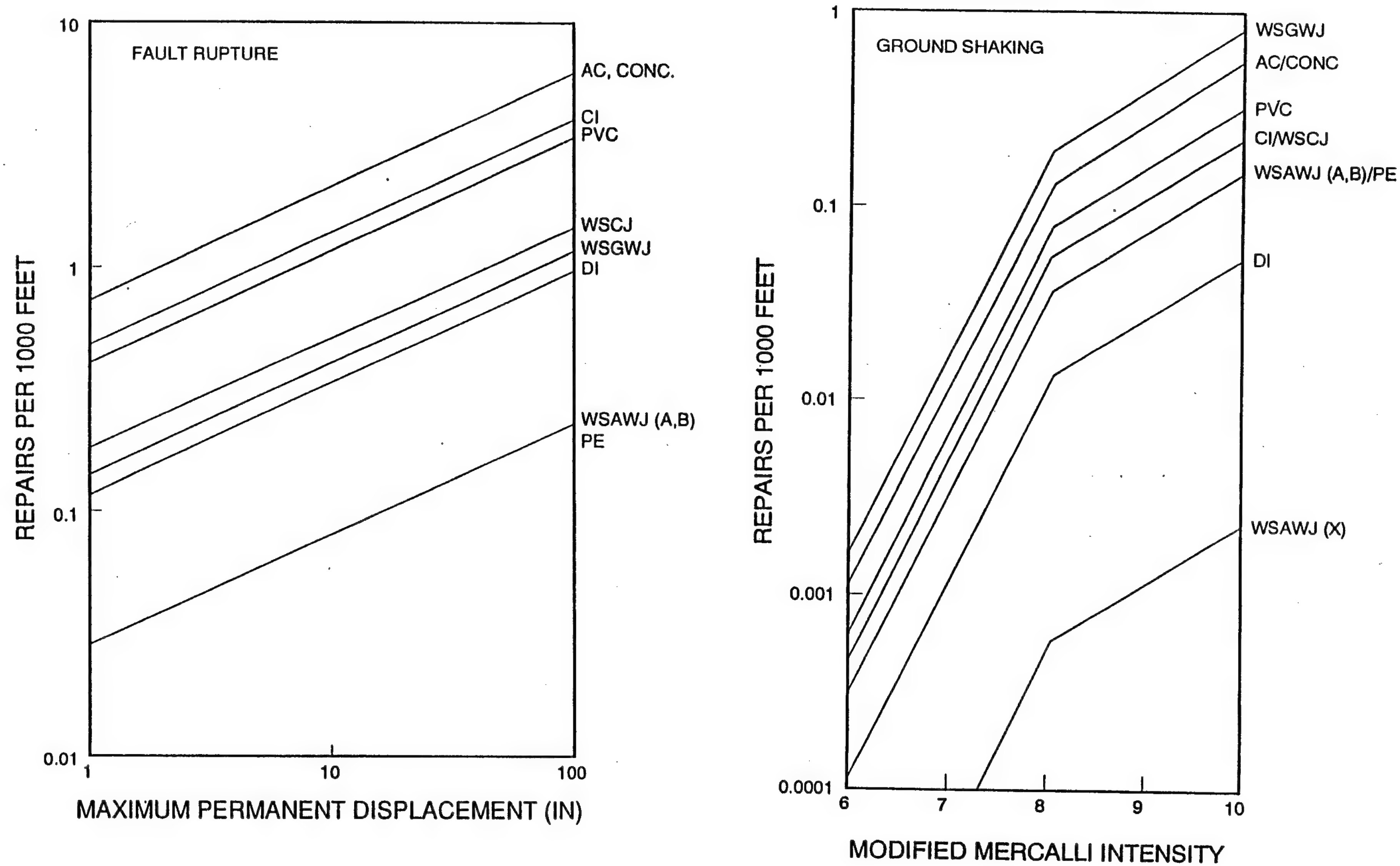
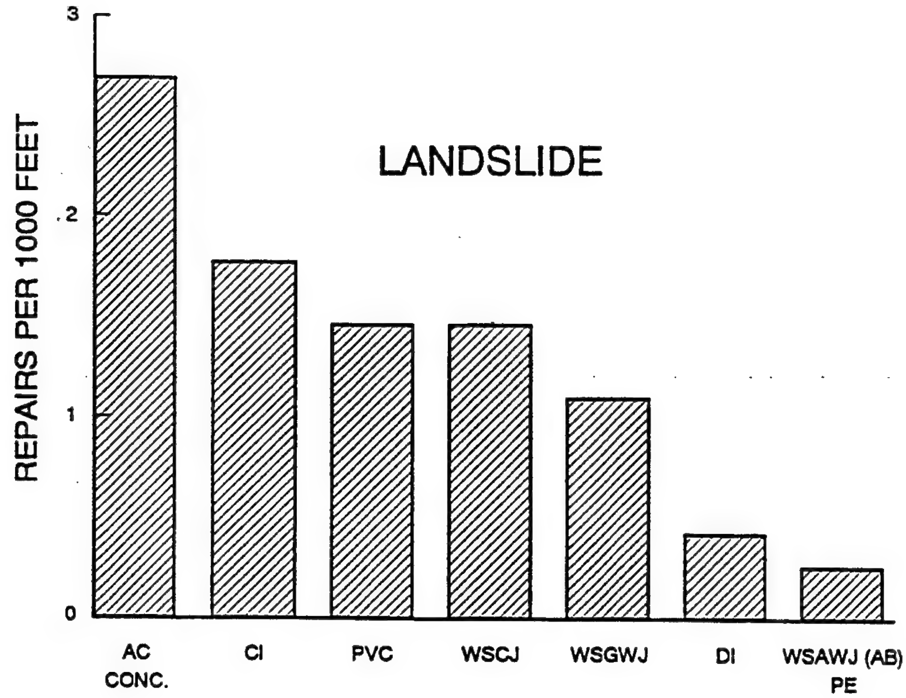


Figure 5-4. Pipe repair model for fault rupture and shaking, from Eguchi et al. (1994).



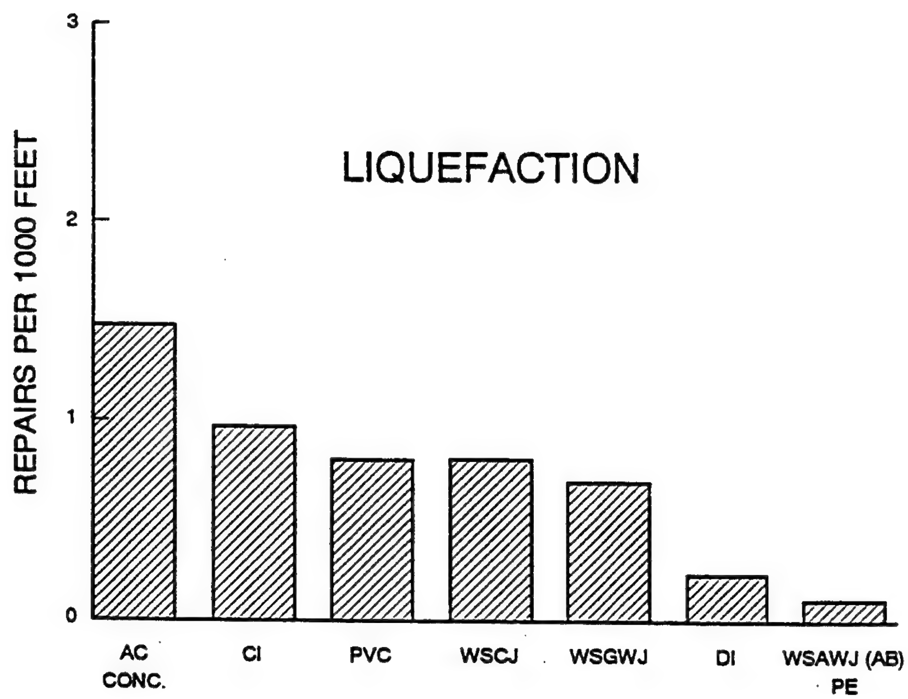
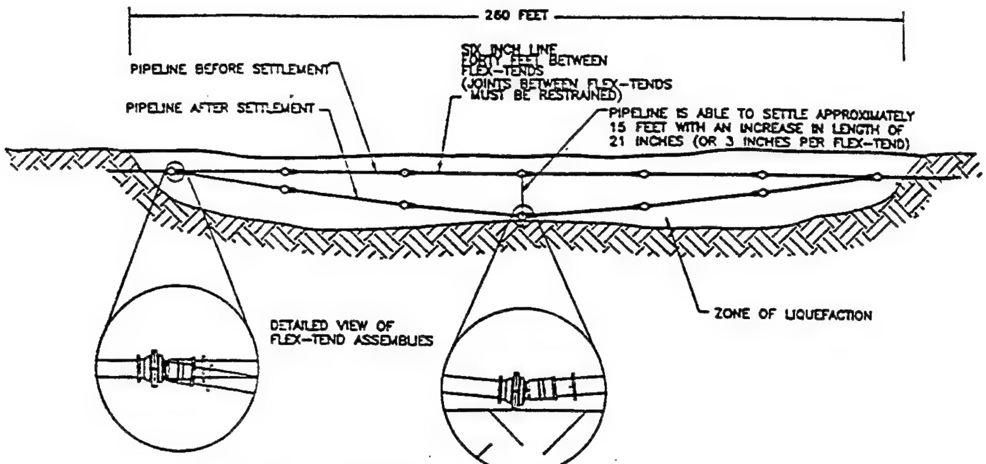
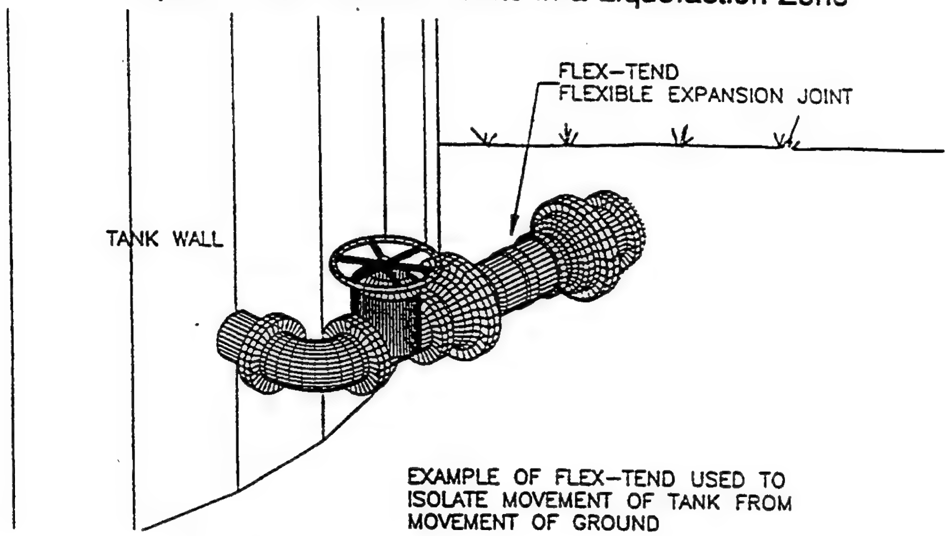


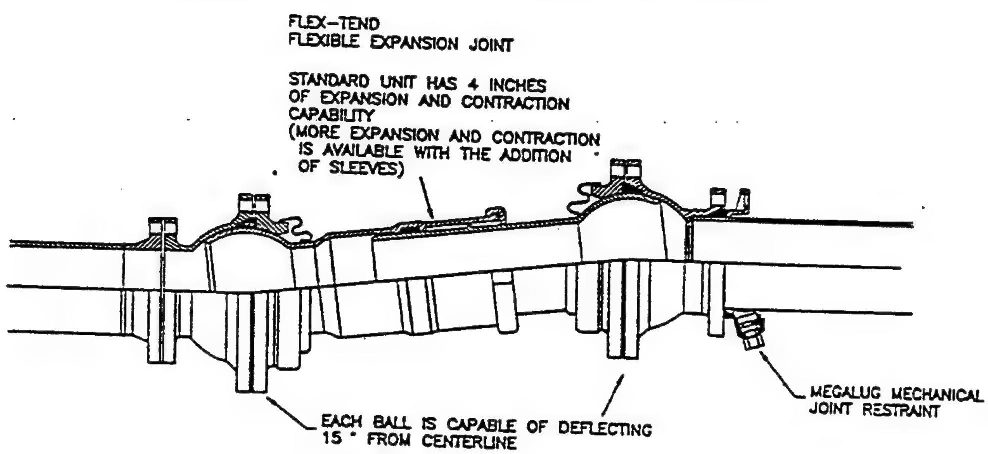
Figure 5-5. Pipe repair model for landslide and liquefaction, from Eguchi et al. (1994).



Pipeline with Flexible Joints in a Liquefaction Zone



Flexible Joint for Pipe-Structure Connection



A Flexible Joint Cross Section

Figure 5-6. Flexible pipe connections.

(1) *General* The provisions of this paragraph apply to the following: ....

(b) Horizontal pipe. All horizontal pipes and attached valves. For the seismic analysis of horizontal pipes, the equivalent static force will be considered to act concurrently with the full dead load of the pipe, including contents.

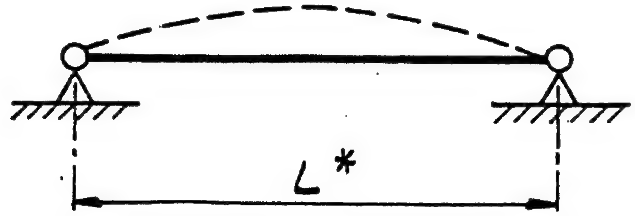
(c) Connections. All connections and brackets for pipe will be designed to resist concurrent dead and equivalent static forces. The seismic forces will be determined from the appropriate provisions below. Supports will be provided at all pipe joints unless continuity is maintained. See paragraph (4) below for acceptable sway bracing details.

(d) Flexible couplings and expansion joints. Flexible couplings will be provided at the bottoms of risers for pipes larger than 3½ inches in diameter. Flexible couplings and expansion joints will be braced laterally unless such lateral bracing will interfere with the action of the flexible coupling or expansion joint. When pipes enter buildings, flexible couplings will be provided to allow for relative movement between soil and building.

(e) Spreaders. Spreaders will be provided at appropriate intervals to separate adjacent pipe lines unless the pipe spans and the clear distance between pipes are sufficient to prevent contact between the pipes during an earthquake.

(2) *Rigid and rigidly attached piping Systems.* Rigid and rigidly attached pipes will be designed in accordance with paragraph 12-3. The equivalent static lateral force is given by $F_p = ZI_pC_pW_p$ (SEAOC eq 1-10), where C_p is equal to 0.75 and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. A piping system is assumed rigid if the maximum period of vibration is 0.05 second (for pipes that are not rigid see paragraph (3) below). Figures 12-4, 12-5, and 12-6, (Shown in this report as Figures 5-7, 8 and 9) which are based on water-filled pipes with periods equal to 0.05 second, are to be used to determine the allowable span-diameter relationship for Zones 1, 2, 3, and 4 for standard (40S) pipe; extra strong (80S) pipe; Types K, L, and M copper tubing; and 85 red brass or SPS copper pipe.

(3) *Flexible piping Systems.* Piping systems that are not in accordance with the rigidity requirements of paragraph 12-7c(2) (i.e., period less than 0.05 second) will be considered to be flexible (i.e., period greater than 0.05 second). Flexible piping systems will be designed for seismic forces with consideration given to both the dynamic properties of the piping system and the building or structure in



DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	6'-6"	6'-6"	5'-0"	4'-9"	4'-6"	5'-6"
1½	7'-6"	7'-9"	5'-9"	5'-6"	5'-6"	6'-6"
2	8'-6"	8'-6"	6'-6"	6'-6"	6'-3"	7'-0"
2½	9'-3"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
3	10'-3"	10'-6"	7'-9"	7'-6"	7'-6"	8'-9"
3½	11'-0"	11'-0"	8'-3"	8'-3"	8'-0"	9'-3"
4	11'-6"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
5	12'-9"	13'-0"	10'-0"	9'-6"	9'-6"	10'-9"
6	13'-9"	14'-0"	10'-9"	10'-6"	10'-3"	11'-6"
8	15'-6"	16'-0"				
10	17'-0"	17'-6"				
12	18'-3"	19'-0"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

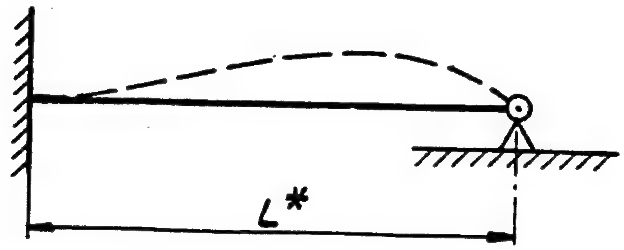
$L^2 = 0.50\pi T_a \sqrt{EIg/w}$

E = MODULUS OF ELASTICITY OF PIPE

I = MOMENT OF INERTIA OF PIPE

w = WEIGHT PER UNIT LENGTH OF PIPE AND WATER

Figure 5-7. Maximum Span for rigid pipe, pinned-pinned.
From NAVFAC P355 Figure 12-4

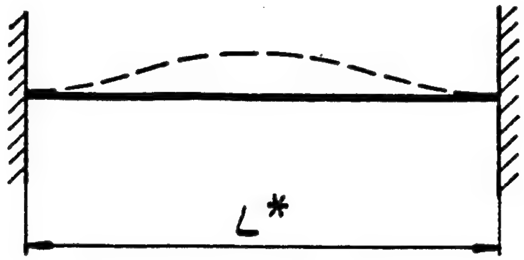


DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	8'-0"	8'-0"	6'-0"	6'-0"	5'-9"	6'-9"
1½	9'-6"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
2	10'-6"	10'-9"	8'-0"	8'-0"	8'-9"	9'-0"
2½	11'-9"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
3	12'-9"	13'-0"	9'-9"	9'-6"	9'-3"	10'-9"
3½	13'-6"	14'-0"	10'-6"	10'-3"	10'-0"	11'-6"
4	14'-6"	14'-9"	11'-0"	11'-0"	10'-9"	12'-3"
5	16'-0"	16'-3"	12'-3"	12'-0"	11'-9"	13'-3"
6	17'-0"	17'-9"	13'-6"	13'-0"	12'-9"	14'-3"
8	19'-3"	20'-0"				
10	21'-3"	22'-0"				
12	23'-0"	23'-6"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 0.78\pi T \sqrt{EIg/w}$$

Figure 5-8. Maximum Span for rigid pipe, fixed-pinned. From NAVFAC P355 Figure 12-5



DIAMETER INCHES	STD. WT. STEEL PIPE 40S	EX. STRONG STEEL PIPE 80S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	9'-6"	9'-6"	7'-3"	7'-3"	7'-0"	8'-0"
1½	11'-6"	11'-6"	8'-6"	8'-6"	8'-3"	9'-9"
2	12'-9"	13'-0"	9'-9"	9'-6"	9'-6"	10'-9"
2½	14'-0"	14'-3"	10'-9"	10'-6"	10'-6"	11'-9"
3	15'-6"	15'-9"	11'-9"	11'-6"	11'-3"	13'-0"
3½	16'-6"	16'-9"	12'-6"	12'-3"	12'-0"	14'-0"
4	17'-3"	17'-9"	13'-6"	13'-0"	13'-0"	14'-9"
5	19'-0"	19'-6"	15'-0"	14'-6"	14'-3"	16'-0"
6	20'-9"	21'-3"	16'-3"	15'-9"	15'-6"	17'-3"
8	23'-3"	24'-3"				
10	25'-9"	26'-6"				
12	27'-6"	28'-6"				

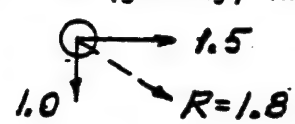
* MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 1.125\pi T_a \sqrt{EI g/w}$$

Figure 5-9. Maximum Span for rigid pipe, fixed-fixed.
From NAVFAC P355 Figure 12-6

Diameter (in.)	Std. Wgt. Steel Pipe - 40S		Ex. Strong Steel Pipe - 80S		Copper Tube Type L	
	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)
1	22	70	22	80	11	17
1-1/2	25	140	26	180	12	35
2	29	220	30	290	14	70
2-1/2	32	380	33	460	15	110
3	34	550	35	710	17	150
3-1/2	36	730	38	930	18	220
4	39	960	40	1,200	19	300
5	41	1,440	44	1,900	20	470
6	45	2,120	46	2,750	22	730
8	49	3,740	54	5,150	26	1,550
10	54	6,080	59	7,670	28	2,620
12	58	8,560	61	10,350	31	3,950

*Maximum spans (L) between lateral supports of flexible piping are based on the resultant of an assumed loading of 1.5 w($ZI_pA_pC_p$ = 1.5) in the horizontal direction and 1.0 w (gravity) in the vertical direction. The resultant is 1.8 w.



The assumed maximum stress is 20,000 p.s.i. for steel and 7,000 p.s.i. for copper. Simple spans (pinned-pinned) are assumed. The calculated maximum lateral displacements are 3.5 inches for steel (E = 29 x 10^6 p.s.i.) and 0.6 inch for copper (E = 15 x 10^6 p.s.i.).

†The horizontal force (F) on the brace is based on 1.5 w L for the maximum span. For shorter spans, $F_{design} = (L_{design}/L)F$.

Figure 12-7. Maximum span for flexible pipes in Seismic Zone 4.

Zone	L (feet)	F (pounds)	$ZI_pA_pC_p$
3	1.1	0.8	1.12
2B	1.20	0.6	0.75
2A	1.25	0.5	0.56
1	1.35	0.3	0.28

Table 12-2. Multiplication factors for figure 12-7 for Seismic Zones 1, 2, and 3 or for cases where $ZI_pA_pC_p$ is not equal to 1.5.

Figure 5-10. NAVFAC P355 Figure 12-7 and Table 12-2.

which it is placed. In lieu of a more detailed analysis, the equivalent static lateral force is given by $F = ZI_pA_pC_pW_p$ (eq 12-2), where $A_p = 5.0$, $C = 0.75$, and is the weight of the pipes, the contents of the pipes, and the attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. Figure 12-7 (Shown in this report as Figure 5-10) may be used to determine maximum spans between lateral supports for flexible piping systems. The values are based on Zone 4 water-filled pipes with no attachments. If the weight of the attachments is greater than 10 percent of the weight of the pipe, the attachments will be separately braced, or substantiating calculations will be required. Temperature stresses have not been considered in Figure 12-7 (Figure 5-10 herein). If temperature stresses are appreciable, substantiating calculations will be required.

(a) *Use of Figure 12-7.* The maximum spans and design forces were developed for $ZI_pA_pC_p = 1.50$. For lower $ZI_pA_pC_p$ values, the spans and forces may be adjusted by the values in Table 12-2. (Figure 12-7 and Table 12-2 are reproduced in this report as Figure 5-10)

(b) Separation between pipes. Separation will be a minimum of four times the calculated maximum displacement due to F_p, but *not less* than 4 inches clear between parallel pipes, unless spreaders are provided ...).

(c) Clearance. Clearance from walls or rigid elements will be a minimum of three times the calculated displacement due to F_p, but not less than 3 inches clear from rigid elements.

(4) Alternative method for flexible piping systems. If the provisions in the above paragraphs appear to be too severe for an economical design, alternative methods based on the rationale described in paragraph 12-4 and paragraph 12-8 may be applied to flexible piping systems.

Figure 5-11 shows acceptable details for sway bracing from NAVFAC P355.

NAVFAC P355 Chapter 14 has several figures which illustrate good engineering practice for pipelines. Figure 14-1 from NAVFAC P355 shows a sewer manhole in which the bell is located at the manhole and encased in concrete to increase its strength while still providing flexibility to the mating pipe. When a pipeline passes through a wall good practice allows a 2 foot square space in the wall around the pipe; the space is filled with oakum or other expandable material to provide for differential movement. Good practice provides flexible couplings at both ends of a 90 degree bend and on each of the three sides of a tee connection. The manual suggests that prudent planning take into account the possible loss of electrical power to pumps and the potential need for manual operation of fuel pumps and backup lighting during an emergency. A properly designed

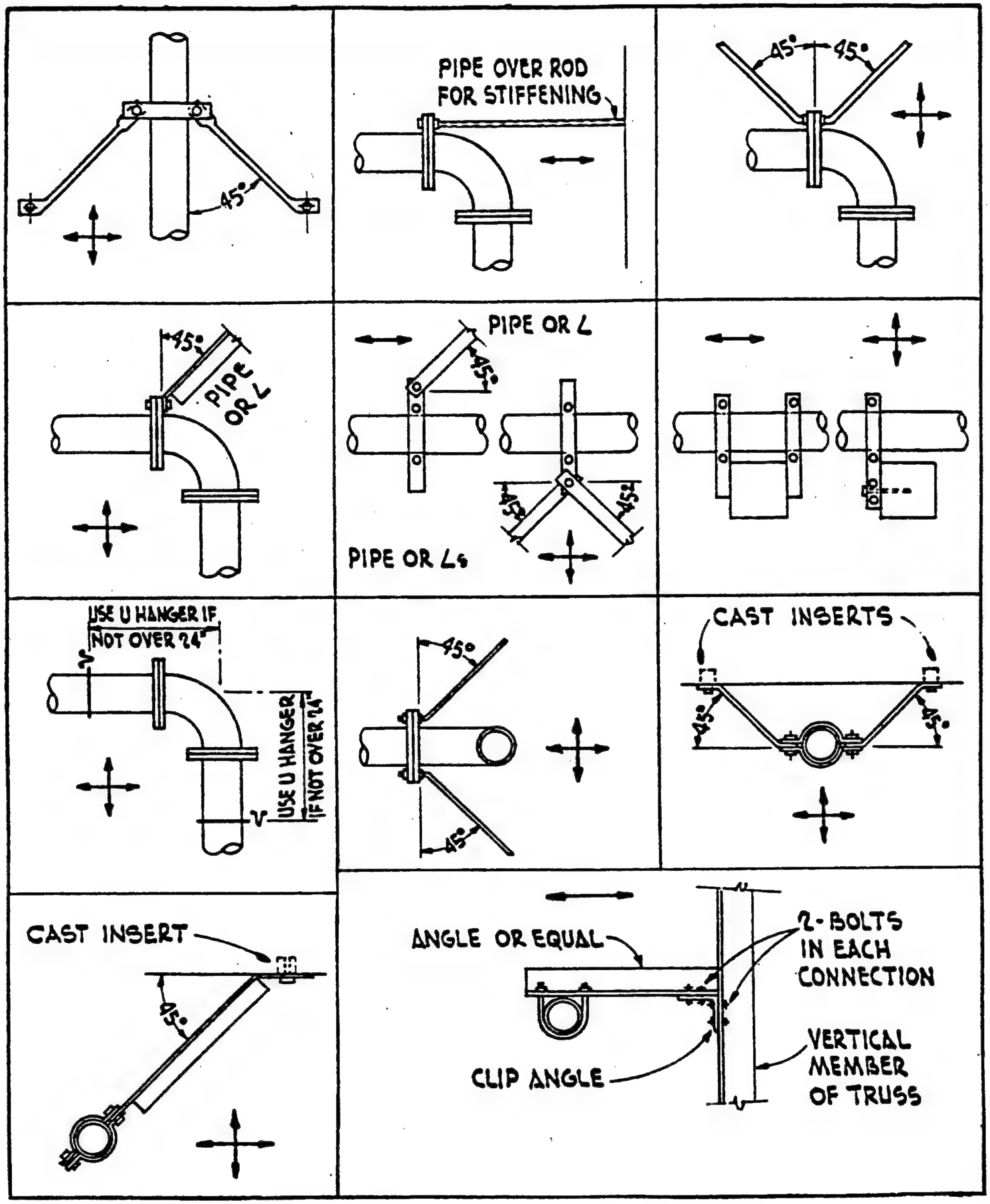


Figure 5- 11. Acceptable seismic details for sway bracing from NAVFAC P355 Figure 12-8.

pipeline distribution system will include alternative routes and valves to isolate potential pipe breaks and maintain operation with improved reliability.

The following are taken from Chapter 14 and are required in these criteria:

- No section of pipe in Zone 2, 3 or 4 shall be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement unless the pipe is shown to have sufficient stress capacity.

- When secondary or standby gas supply systems cannot be justified for a site, gas distribution networks for buildings in Zones 2, 3 and 4 housing essential functions dependent upon gas shall include an above ground valved and capped stub. Provision will be made for attachment of a portable, commercial-sized gas cylinder system to this stub

- For essential facilities in Seismic Zones 3 and 4, an earthquake activated gas shutoff valve shall be provided. If an earthquake activated shut-off valve presents the possibility of disrupted service in the buildings where the fire hazard is small, manually operated valves shall be installed.

- Buildings housing essential functions shall be provided with two or more water service lines connected to separate sections of the supply grid to minimize loss of service. Service shall be interconnected within the building by check valves to prevent backflow.

- Flexible connections shall be used between valves and lines for valve installation on pipes 3 inches or larger in diameter.

- Flexibility shall be provided by use of flexible joints or couplings on a buried pipe passing through different soils with widely different degrees of consolidation immediately adjacent to both sides of the surface separating the different soils.

- Flexibility shall be provided by use of flexible joints or couplings at all points that can be considered to act as anchors and at all points of abrupt change in direction and at all tees.

- NAVFAC P355 paragraph 12.-7 (cited above) specifies restraints for critical piping in essential facilities.

Piping containing hazardous materials shall contain numerous valves and check valves to minimize release of materials if there is a break. A secondary containment system should be incorporated where feasible. When piping is connected to equipment or tanks, use of braided flexible hoses is preferable to bellow-type flexible connectors since the latter has been noted to fail from metal fatigue. Welded joints are preferable to threaded or flanged

joints. If flanged joints can not be avoided the use of self-energizing or spiral wound gaskets can allow a bolt to relax wile continuing to provide a seal, Association of Bay Area Governments (1990). Seismic shutoff valves should be used where necessary to control a system or process. These systems can be triggered by a mechanical sensor on the valve or by a remote electronic sensor which can control a number of elements. Choice of valves should be restricted to approved valves to reduce leakage after closure.

New Tanks

Understanding the individual failure modes of individual tank components such as piping, restraints, tie down anchors, piles etc. is an important part of the development of a comprehensive design specification for quality performance. In general there are several types of tanks in use. Flat bottom vertical tanks vary in size from small 10,000 gallons to over several million gallons. Tanks can be anchored or unanchored. Large tanks can have internal columns to support the roof. Vertical tanks can be placed on a prepared mat or a ring foundation along the tank perimeter with or without edge confinement. Horizontal tanks are of cylindrical shape and are usually supported on two saddles. The typically range in size from 100 gallons to 10,000 gallons. Smaller tanks can be supported on legs. Typically fuel tanks for portable generators are of this type. Summers (1997) lists major causes of tank failure and includes the following:

- Buckling of the tank wall (termed elephant foot buckling)
- Breakage of inlet/outlet piping from uplift
- Tearing of the tank wall at discontinuities
- Tearing of tank wall from overconstrained stairways between the foundation and tank shell
- Roof damage caused by sloshing
- Foundation failures and liquefaction

Schiff (1991) presents a summary of observed damage to tanks. Flatbottom vertical tanks tend to be most vulnerable to earthquakes, especially tanks with a large liquid depth to radius ratio. One failure mode of the tank is buckling and is caused by rocking of the tank or differential settlement of the foundation under the tank Unanchored tanks with a radius-to-wall thickness of over 600 have been damage most often. These tanks develop sufficient overturning moment to cause the edge to lift off the ground. The opposite side sustains high compressive stresses which cause bulging at the base. Summers (1997) reports that a 100-foot diameter tank 30 feet high sustained 14 inches of uplift in the 1971 San Fernando earthquake. Similar tank behavior was noted in the 1989 Loma Prieta earthquake with an observation of 6 to 8 inches of uplift and 18 inches in the 1964 Alaska earthquake. Even anchored tanks can fail in this manner if there is anchorage failure. Generally a pattern of well distributed anchor bolts works best compared with fewer larger bolts. Maintenance is requirement to inspect the condition of the anchor bolts and replace those with corrosion.

Vertical motion can cause local tensile membrane deformation, elephant foot bulging, at the base of the tank. This can also be induced by rocking. It is interesting to note the annular volume of the bulge is about equal to the earthquake vertical displacement times the tank area. It is postulated that the fluid has high inertia and the increase in fluid pressure from the vertical component of the earthquake causes the perfectly symmetrical bulge. Increasing the wall thickness may reduce the occurrence, but might simply result in the buckling occurring high up on the tank. The weld between the tank base plate and side wall has also been observed to fail. This is caused by uplift forces and is often associated with corrosion induced weakness. A failure of the weld can open a portion of the seam causing rapid loss of contents and a partial vacuum in the tank causing internal buckling. Tank venting is important to restrict implosion.

Small unanchored tanks less than 30 feet in diameter have been observed to slide on their foundation. In tanks which are full, sloshing can cause roof and upper wall failures. As noted liquefaction was a major cause of the extent of waterfront damage and can cause settlement and lateral spreading.

The primary failure mode of horizontal tanks is anchorage failure or inadequate anchorage which causes tank slippage off the saddles. Typically the tank is fully anchored only on one side to allow for expansion. The single restraint must be capable of withstanding horizontal, vertical and torsional components of motion. The saddle must be designed to resist forces acting on its weak axis as well. Elevated fuel tanks often fail by buckling of the supports. These tanks stands require adequate tie-down and diagonal bracing

Water tanks tend to be kept full however hydrocarbon tanks tend to be half-full and sloshing must be considered. Lack of tank venting has resulted in implosion. Anchor bolts embedded in concrete used for tank uplift restraining must have sufficient concrete confinement to prevent pullout. Shear reinforcing should be used to provided needed concrete confinement to prevent anchor bolt failure. Typically anchor bolts for new construction are designed with a safety factor of 4; a value of 3.0 is used for evaluation of existing anchors. Provisions must be made to evaluate the effect of corrosion in reducing the strength of existing construction.

To achieve the required system performance and satisfy regulations additional backup hazardous material containment systems are used. Containment systems are composed of either a singular system or a dual system as mandated by public law discussed in the Criteria. A singular system provides only a single structural element system for material containment. Singular systems are restricted to small systems of less than 660 gallons such that a failure shall not produce catastrophic damage. A dual system is composed of a primary containment structure and a secondary containment system which shall function should the primary system be damaged. Containment systems open to rain shall need to be drained.

Design of tanks shall utilize the API 650 procedures discussed above.

Tanks shall be designed against sliding and uplift and be fully anchored. The height of sloshing may be calculated using an equation by Wozniak and Mitchel (1978). This height should be used for freeboard calculations associated with roof damage. The hydrodynamic forces which create overturning moments also act on the foundation and must be taken into account in foundation design.

Essential tanks shall be designed to resist Level 3 earthquakes using response spectra and the API 650 procedures.

For both ordinary and essential tanks, a requirement exists to prevent uncontrolled loss of contents and pollution of the environment an event for a Level 4 event. Such requirements shall be met by provision of a containment system. Singular systems must be designed so that the structure itself provides the margin of safety to preclude release of materials. Dual systems may be evaluated on the basis of total system performance allowing for the presence of the secondary confinement, such that any release from the primary containment is confined within the secondary containment. The secondary containment must function at such a level so as not to permit an unacceptable release of materials.

Failure of pipe to tank connections is common when there is insufficient flexibility to accommodate differential motion between the tank and pipe network. This can be prevented by having the first pipe anchor point at a sufficient distance (15 pipe diameters minimum) from the edge of the tank and the pipe oriented in a radial direction away from the tank. Flexible connections and expansion joints can accommodate differential motion provided they are sized properly. The most important element of seismic design of pipelines is proper siting. It is imperative to avoid areas of lateral spreading and landslide. Additionally stairways should not be attached to both the foundation and the tank wall.

Summers (1997) presents the following information:

> Tank uplift during earthquakes can damage attached piping and other appurtenances. ...... anchored tank appurtenances may he designed for some level of anchor bolt stretch. A value of 2 inches is proposed in the latest NEHRP provisions (BSSC, 1994).
>
> API 650 states that piping attached to the tank bottom that is not free to move vertically shall be placed a radial distance from the shell/bottom connection of 12 inches greater than the uplift length predicted by the API 650 uplift model. The API 650 uplift model, however, may underpredict the amount of radial uplift (Manos, 1987; Dowling and Summers, 1993). It may be prudent to consider changing this requirement to... twice the API 650 model.......

> Walkways between tanks should he designed to accommodate relative movement of the tanks. ..... In lieu of a more rigorous analysis, a walkway should he designed to accommodate a total of 12 to 18 inches of movement, at least in the zones of high seismicity.
>
> Attached ringwalls should be designed appropriately. Anchoring a tank to a small ringwall and not developing the forces into the soil by the weight of the ringwall or with piles should he viewed with caution. Anchor bolts need to be designed such that they behave in a ductile manner, both in terms of the force transfer to the shell and pullout from the concrete foundation.

Existing Tanks

Tanks built prior to the late 1970s probably lack consideration of seismic design since it was during that period in which code provisions were first implemented. However the provisions in API and AWWA are generally thought to be conservative such that evaluation of existing tanks by the new tank criteria may unduly penalize them. Summers (1997) reports the following:

> There are several alternatives to the API methodology that might be considered for use in evaluation of existing tanks. One such method is a modified version (Dowling and Summers, 1993; Summers and Hults, 1994; and ASCE Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, 1997) of a method developed by George Manos (Manos, 1987) presented herein Manos' method is based on experimental studies, as well as on observed behavior of unanchored tanks during past earthquakes. Instead of trying to model the complex uplifting plate behavior, Manos assumes a stress distribution at which the shell will buckle and solves for the resisting moment produced by the sum of the stresses. This resisting moment can then be compared to the overturning moment and the resisting acceleration solved for.
>
> The method proposed herein for evaluation of unanchored storage tanks is based on that of Manos, but includes some important variations. The most notable of these are (Dowling and Summers, 1993):
>
> a. Tank anchorage is recommended in zones of high seismicity whenever the ratio of safe operating height to tank diameter exceeds two. Based on the data presented in Manos, and the higher level of risk for taller tanks, this is believed to be the upper limit of applicability of the Manos method.

b. The allowable compressive stress in the tank shell should not exceed 75% of the theoretical buckling stress, as presented in Manos, nor should it exceed the material yield strength. This last requirement is significant for thicker-walled tanks. Note that an increase in the allowable compressive stress beyond 75% of the theoretical buckling stress may be justified under certain circumstances.

c. The compressive force in the tank shell should not exceed the total weight of the fluid contents. This has the effect of imposing an upper bound on the resisting moment.

A comparison of the results of an evaluation of a 35 ft diameter, 30 ft high tank in a high seismic zone filled to a height of 26 ft 4 in, using the modified Manos and API methodologies, ... (was made. The) API approach would require either a reduction in fill height by about 40% to 16 ft 6 in or tank anchorage, whereas the modified Manos method indicates that the seismic safe operating height can be increased to 20 ft 1 in. Hence, the required reduction is reduced from 9 ft 10 in to 6 ft 3 in, and the benefit is immediately apparent.

The Manos (1987) develops the following relationships for the compressive member stress distribution near the tank bottom as:

$$\sigma_{max} = 0.75\ \sigma_{cl}$$

where

$$\sigma_{cl} = \frac{E\,t_s}{R\sqrt{3(1-\upsilon^2)}}$$

$$\sigma = \sigma_{max}\cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

if $\upsilon = 0.3$

$$\sigma = 0.46\frac{Et_s}{R}\cdot\cos\left(\frac{\pi\phi}{2\phi_0}\right)$$

$$\phi_0 = 0.65\ S\left(\frac{R}{H}\right)^n\left(\frac{t_s}{t_p}\right)^{0.1}$$

$$n = 0.1 + 0.2\frac{H}{R} \leq 0.25$$

where

E	Young's modulus
H	Liquid height
R	Tank radius
S	Foundation coefficient
t_s	tank wall thickness
t_p	tank bottom plate thickness
υ	Poisson's ratio
σ	axial member stress
σ_{cl}	Theoretical buckling stress

Manos develops an empirical expression for the limiting impulsive acceleration capacity of a tank, C_{eq}, as

$$C_{eq} = \frac{0.372}{\rho_w} \frac{S\,E\,t_s^2}{G\,R\,H^2} \frac{m_t}{m_l} \left(\frac{R}{H}\right)^n \left(\frac{t_s}{t_p}\right)^{0.1}$$

where

G	Liquid density ratio
m_l	Liquid impulsive mass
m_t	Total liquid mass
ρ_w	Density of water or tank liquid

Manos compares the acceleration capacity to the applied acceleration which is based on a tank response spectrum determined from an amplified ground motion spectrum between the periods of 2 and 9 seconds having 2 percent damping. He proposes a 4.3 acceleration amplification factor to be applied to the ground motion spectrum as a conservative approximation of structure amplification. The acceleration capacity must be larger than the acceleration demand.

Special Drainage for Petroleum Offloading and Fueling Piers.

The following requirement shall be adhered to pertaining to special drainage for petroleum offloading and fueling piers:

a) An intercept system shall be required to collect oil spills. In normal operation, deck drainage shall outfall through the sump pumps into the harbor. If an oil spill occurs, pressing a deck-mounted button shall close a motor-operated outfall valve and start the sump pumps which pump the spill to a collection point. when the spill drainage procedure is completed and all oil is removed from the system, the system shall revert to normal operation.

b) Contaminated rainwater runoff of all deck drainage due to contact with residual drippings on the deck shall be collected.

Utilities On Piers

Piers may contain pipelines for freshwater, saltwater, steam, compressed air, waste oil, sewer, fuels, as well as electrical power and communication lines. Ship demands dictate the configuration. In general design of these lines follows the general provisions discussed herein. It is essential that the lines be attached to the supporting structure with sufficient rigidity that the lines are restrained against independent movement. Attachments to a pier may be analyzed as simple two-degree-of freedom systems as discussed in NAVFAC P355, Chapter 12. Resonance amplification can occur when the natural period of the supported pipe is close to the fundamental period of the pier structure. Flexible connections/sections shall be used to bridge across expansion joints or other locations where needed. All piping and utility lines on a pier shall be designed as essential construction. Specifically, the provisions of NAVFAC P355 Section 12-7d shall be used. Section 12-7d is discussed above under pipelines. Pipelines containing hazardous materials may have to be of double wall construction based on requirements of local environmental requirements. Check-valves should be used to minimize the loss of contents to minimize the size of a spill if there is a pipeline break.

Electric Power

A typical electric power system includes transformer and distribution lines, local transformers and backup generators. Linkages exist between electric power and other lifelines; for example, electricity is needed for pumps to maintain pressure in water distribution systems. Most damage has resulted from overturning or sliding of inadequately anchored or braced components. Often electrical equipment is situated on top of poles or supports undergoes extensive displacements rupturing attached cables. Pole mounted transformers are supported on raised platforms; typically they are not secured to the platform.

Inadequately mounted transformers have been observed to fall from pedestals causing major damage to bushings, radiators and internal parts. An alternative failure mode is excessive sliding without overturning. Sliding breaks the rigid bus connections, the lightning arrestors and insulators. Past practice had transformers mounted on rails anchored in concrete slabs. When these mounts failed, extensive damage resulted. Schiff (1991) suggests that new installation design use concrete pads with steel anchor plates securely embedded in the pad and flush with the surface. The transformer is welded to the anchor plate eliminating the need for an intricate pattern of tie down bolts. Spare

transformers were kept unsecured for relocation as needed, which can overturn. All transformers whether in service or spare require the same restraint.

Criteria for electrical power lifelines focuses on providing adequate anchorage. All transformers on poles or platforms shall be anchored against overturning or sliding. All equipment shall be anchored as required. Equipment deemed as of ordinary importance shall have lateral force requirements based on provisions of the 1994 Uniform Building Code and NAVFAC P355. Equipment deemed as essential shall have the lateral force requirement based on local site conditions using peak ground acceleration for essential level facilities and a response spectra. In any case lateral forces shall not be less than Code provisions with an importance factor for essential structures/components . This resulting force shall be used as a substitute for Code forces and all remaining Code provisions will apply.

Telecommunications Lifelines

Telecommunications encompasses conventional telephone requirements, communication systems, and equipment control lines. The equipment must be rugged enough to withstand the shaking. The IEEE has established fragility requirements for some equipment found in nuclear power plants. Some other types of equipment also have fragility data. The equipment must be attached in a manner to prevent damage. Attachment can be made by rigidly securing the item against overturning and sliding or where the equipment is delicate it may be mounted on isolators to reduce transmitted motions. A variation of both approaches consists of leaving a large piece of equipment free to slide within restrained limits. This limits the shaking motion which can be transmitted to the equipment by allowing sliding to occur; elastic bumpers limit the range of motion. Obviously the equipment must have an aspect ratio to preclude overturning.

Traditional damage has included overturning of cabinet mounted electronics, failures of suspended ceilings, rupture of piping and water damage to equipment, rupture of cables connecting equipment which became dislodged, weld failures, and inadequate sizing of restraints. Design calculations must consider the inertia force of an object in overturning and sliding. Elements attached to the structure must consider the relative displacement between anchorage points. Flexible supports must consider resonance points where the period of vibration of the flexible mount is the same as that of the structure; stiffening the mount can eliminate resonance.

Required support equipment generally includes backup power generators, uninterruptible power supplies, emergency lighting, voltage controllers, etc. and may also include air-conditioning units, halon firefighting systems etc.

Calculation Of Lateral Force Requirements

The 1997 NEHRP FEMA 273 provisions calculate seismic forces as:

$$F_p = 1.6\, S_{XS}\, I_P\, W_p$$

Alternatively F_p may be calculated by:

$$F_p = 0.4\, a_p\, S_{XS}\, I_P\, W_p\, (1 + 2x/h)\ /\ R_p$$

F_p calculated by the second equation need not exceed F_p calculated by the first equation but a minimum F_p is given by:

$$F_{p\ minimum} = 0.3\, S_{XS}\, I_P\, W_p$$

where:

- a_p — Component amplification factor that varies from 1.00 to 2.50 (select appropriate value from FEMA 273 Table 11-2).
- F_p — Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.
- S_{XS} — Spectral response acceleration at short periods
- I_p — Component importance factor that is either 1.00 or 1.50
- W_p — Component operating weight.
- R_p — Component response modification factor that varies from 1.25 to 6.00 (select appropriate value from FEMA 273 Table 11-2).
- x — Elevation in structure of component relative to grade elevation.
- h — Average of elevation of structure relative to grade elevation.

The force F_p shall be applied independently vertically, longitudinally, and laterally in combination with service loads associated with the component. When positive and negative wind loads exceed F_p for nonbearing exterior wall, these wind loads shall govern the design. Similarly when the building code horizontal loads exceed for interior partitions, these building code loads

Evaluation Of Above Ground Piping Systems

The following is taken essentially verbatim from "Proposed Guidance for California Accidental Release Prevention Program Seismic Assessments" (1998).

Evaluation of piping systems are primarily accomplished by field walkthroughs. Such qualitative evaluations of piping systems are best done by an engineer experienced in this area, visually inspecting the piping system under concern. This is preferred because some piping is field routed and in some instances, piping and supports have been modified from that shown on design drawings. This guidance is primarily intended for ductile steel pipe constructed to a national standard. Evaluation of other piping material is also discussed below.

The procedure for evaluating above ground piping systems should be as follows:

1) Identify piping systems to be evaluated.

2) Determine original design code basis and materials of construction, to the extent possible.

3) Assess extent of obvious corrosion/erosion.

4) Perform a walkthrough of the piping systems for seismic capability. Document the walkthrough and identify areas for detailed evaluation.

5) Complete the detailed evaluation of any identified areas and recommend remedial actions.

Damage to or failure of pipe supports should not be construed as a piping failure unless it directly contributes to a pressure boundary failure. The intention here is to preserve the essential pressure containing integrity of the piping system but not necessarily leak tightness. Therefore, this procedure does not preclude the possibility of small leaks at bolted flanged joints. Ductile piping systems have, in general, performed adequately in past earthquakes. Where damage has occurred, it has been related to the following aspects of piping systems:

1) Excessive seismic anchor movement.

2) Interaction with other elements.

3) Extensive corrosion effects.

4) Non-ductile materials such as cast iron, fiberglass (PVC), glass, etc. combined with high stress or impact conditions.

Seismic anchor movements could result in relative displacements between points of support/attachment of the piping Systems. Such movements include relative displacements between vessels, pipe supports, or main headers for branch lines. Interaction is defined as the seismically induced impact of piping systems with adjacent structures, systems, or components, including the effects of the falling hazards. Corrosion

could result in a weakened pipe cross section that could fail during an earthquake. Additional aspects of piping systems which should also be reviewed during the walkthrough for seismic capability are:

1) Large unsupported segment of pipe,

2) Brittle elements,

3) Threaded connections, flange joints, and special fittings, and

4) Inadequate supports, where an entire system or portion of piping may lose its primary support.

Special features or conditions to illustrate the above concerns include:

1) Inadequate anchorage of attached equipment,

2) Short/rigid spans that cannot accommodate the relative displacement of the supports, e.g., piping spanning between two structural systems,

3) Damaged supports including corrosion,

4) Long vertical runs subject to inter level drift,

5) Large unsupported masses (e.g., valves) attached to the pipe,

6) Flanged and threaded connections in high stress locations,

7) Existing leakage locations (flanges, threads, valves, welds),

8) External corrosion,

9) Inadequate vertical supports and/or insufficient lateral restraints,

10) Welded attachments to thin wall pipe,

11) Excessive seismic displacements of expansion joints,

12) Brittle elements, such as cast iron pipes,

13) Sensitive equipment impact (e.g., control valves), and

14) Potential for fatigue of short to medium length rod hangers which are restrained against rotation at the support end.

The walkthrough is the essential element for seismic evaluations of piping systems. Careful consideration needs to be given to how the piping system will behave during a seismic event, how nearby items will behave during a seismic event (if they can

interact with the piping system) and how the seismic capacity will change over time. The walkthrough should be performed by a licensed engineer familiar with how equipment responds to earthquake loads. Detailed analysis of piping systems should not be the focus of this evaluation. Rather it should be on finding and strengthening weak elements. However, after the walkthrough is performed and if an analysis is deemed necessary, the following general rules should be followed:

1) Friction resistance should not be considered for seismic restraint, except for the following condition: for long straight piping runs with numerous supports, friction in the axial direction may be considered,

2) Spring supports (constant or variable) should not be considered as seismic supports,

3) Unbraced pipelines with short rod hangers can be considered as effective lateral supports if justified,

4) Appropriate stress intensification factors ("i" factors) should be used, and

5) Allowable piping stresses should be reduced to account for fatigue effects due to significant cyclic operational loading conditions. In this case the allowables presented in the next section may need to be reduced.

6) Flange connections should be checked to ensure that high moments do not result in significant leakage.

Procedures for interaction evaluation of piping are as follows:

1) Regulated Substance (RS) piping should be visually inspected to identify potential interactions with adjacent structures, systems, or components. Those interactions which could cause unacceptable damage to piping, piping components (e.g., control valves), or adjacent critical items should be mitigated.

Note that restricting piping seismic movement to preclude interaction may lead to excessive restraint of thermal expansion or inhibit other necessary operational flexibility.

2) The walkthrough should also identify the potential for interaction between adjacent structures, systems or components, and the RS piping being investigated. Those interactions which could cause unacceptable damage to RS piping should be mitigated Note that falling hazards should be considered in this evaluation.

Procedures for corrosion evaluation of piping are as follows:

1) During walkthrough identify conditions conducive to external corrosion.

2) Wall thickness should be evaluated for potential reduction due to erosion or corrosion.

3) Extent of internal corrosion/erosion can be evaluated by any of the following methods:

a. Review of existing corrosion inspection program for RS piping systems,
b. Review of successful operating experience, or
c. Wall thickness measurements.

4) Compare existing corrosion experience and anticipated corrosion to original design corrosion allowance.

The reader is should consult the "Proposed Guidance for California Accidental Release Prevention Program Seismic Assessments" (1998) for additional material not included here such as support design and inertia loads.

Terminal Inspection Team Assessment

A terminal inspection should include a focused study on the relationship of the oil supply system and facility operational and safety requirements. A first step identifies criteria for determining critical functions that require secure energy and selecting those facilities supporting the required safety/operational functions

Post-earthquake recovery efforts must focus on life safety, disaster control and sustaining required operation of essential functions. Recovery efforts must be prioritized to maintain required operations and minimize further damage. It is also essential to relate the repair of utility systems to facility needs since experience has shown that utility system disruptions can produce major impacts upon essential functions.

The elements of the terminal lifeline assessment procedure are to:

- Form an Assessment/Mitigation Team
- Gather information about lifeline utilities
- Determine essential-function requirements from users
- Consider utility outage scenarios
- Assess the vulnerability of required utility lifelines
- Develop mitigation measures

The most significant step in conducting a lifeline assessment is understanding what are the critical operations of the terminal. What critical facilities support these operations and what utilities are critical to support the critical operations? In addition to safe operational requirements are the disaster control and recovery functions which are also highly dependent on lifelines.

The utility/lifeline assessment team must contain a technologist with the ability to understand and assess the terminal's utilities. The team will also need accurate utility system drawings and diagrams for each system being evaluated. Operators and

maintenance personnel can often identify vulnerable points for critical systems. Reports from utility studies, communication system studies, utility contingency plans, security and other plans and studies may provide drawings and analysis that are applicable to this effort. Key hardware components servicing each essential operation need to be identified. Key hardware components are pieces of equipment which must remain operational to support the operation and represent critical links. Utility outage scenarios must be developed based on the seismic potential and the key hardware components accessibility and fragility. The effects of the utility outages must be analyzed including the evaluation of the response, repair, and recovery capability of the oil terminal. Once the team has accomplished the utility systems assessment, they can identify the corrective measures necessary to remedy deficiencies. These remedies are likely to include projects and/or activity operational procedures. The team should rank the mitigation measures and develop plans to implement them.

Analysis

The approach outlined in this section is a guide to assessing the vulnerability of those control systems and utilities which provide service to essential oil terminal operations. The survey team should examine fire detection and suppression, electricity, water, thermal, sanitary and industrial wastewater, compressed air, and communications as well as any other utility. The information gathered is meant to stimulate the thought process and is a tool to assess the utility systems. In gathering these data one will not only compile a comprehensive source of valuable utility information, but will also discover information gaps which might prove critical in an actual utility outage situation. The vulnerability assessment report produced from this information will require rigorous analysis of the particular lifeline utility systems and operating procedures. In analyzing a utility system, it is best to follow a logical pattern from the point of utility supply through the onbase distribution system to the final end-user. In the assessments, provide simplified schematics of each utility system which have key components.

For each outage scenario considered, both area-wide and local utility outages, the impact upon the essential operations should be assessed, the minimum utility requirements needed to support these operations should be determined, and possible corrective measures to meet these requirements should be evaluated. Consideration should be made regarding the interrelationships between utility systems.

Response, Repair, and Recovery Capability

For every key component identified, a comprehensive evaluation the ability to restore the item to affected areas should be conducted. For example, restoration of power may include repair to the disabled component, replacement of the disabled component, actual bypass of that component or provision of backup power. The potential for subsequent problems from the repair, replacement, or bypass should be part of the overall evaluation. Items that should be considered include the following:

1. Availability of spares and replacement equipment (location, time, administrative procedures)
    a. Onsite
    b. Commercial utility
    c. Commercial suppliers
    d. Other sources

2. Availability of equipment needed to effect repairs

    a. Heavy transport equipment
    b. Trucks
    c. Special tools and machinery

3. Availability of personnel

    a. Activity Maintenance Personnel

        1. Crew skill mix
        2. Level of training
        3. Degree of experience
        4. Knowledge of installation systems
        5. Multiple assignments and responsibilities
        6. Staff reassignment

    b. Commercial contractors

        1. Number of contractors available and past relationships
        2. Formal agreements or contracts
        3. Administrative and financial limitations

    c. Commercial utility

        1. Availability potential
        2. Formal and informal agreements
        3. Other commitments or obligations

4. Consideration of adverse working conditions and circumstances

5. Implementation of load shedding/conservation to match available supply

6. Availability of backup generator sets

    a. Verification of operation of generators

b. Maintainability/ability to operate for extended periods
c. Refueling requirements and procedures
d. Ability to relocate and connect to loads

Key Hardware Components

Electrical Distribution System

a. Commercial feeds to activity and their point of origin, point of connection to terminal, and capacities
b. Backup generators

    1. Number of fixed and portable sets assigned
    2. Other generators available on the installation
    3. Size, age, and present assignments

c. All essential loads and all key components utilizing a current wiring diagram
d. Load shedding and any other relevant contingency plans

Water Distribution Systems

Water system key hardware components must include potable water, fire protection water and water requirements related to thermal energy systems (makeup water for boilers and cooling towers).

a. All essential loads and key components shown on current drawings
b. Commercial lines serving the activity and the points of origin
c. Onsite water sources (if any)
d. Onsite water treatment facilities (if any)
e. Capacity and location of storage facilities (if any)
f. Key components dependent on electrical power

    1. Pumps
    2. Water treatment equipment
    3. Valves
    4. Controls

g. Backup electrical generator sets

    1. Number of fixed and portable sets assigned
    2. Size, age, and present assignments

h. Availability of water transport
i. Treatment chemical requirements

Wastewater (Sewage/Industrial Systems)

Wastewater system key hardware components should include collection and treatment facilities for domestic sewage and industrial wastewater. Consider the possible creation of hazards associated with mixing incompatible industrial wastewater streams. Additionally, repairs to sanitary sewers, where waste may be septic, should be accomplished with protective gear and respiratory equipment.

a. Public lines serving the terminal and their point of origin
b. Generating activities
c. Onsite treatment facilities (if any)
d. Storage capacity (i.e., 55 gallon drums, tankers, holding tanks, etc.)
e. Critical functions which generate wastewater
f. Key hardware components from point of generation to treatment/disposal
g. Key components dependent on electrical power

    1. Pumps
    2. Wastewater treatment equipment
    3. Valves
    4. Controls

h. Backup electrical generator sets

    1. Number of fixed generator sets
    2. Size, age and present assignment

i. Availability of transport (i.e., tankers, barges etc.)
j. Treatment chemical requirements

Compressed Air

a. Essential functions requiring compressed air (if any)
b. Sources of compressed air (central, point of use)
c. Key components dependent on electric power
d. Identify compressors capable of operating independent of electrical power
e. Isolation valves
f. Cross connects between distribution lines
g. Backup compressors at critical points of use
h. Backup electrical generator sets

CHECKLIST FOR WALK-THROUGH SCREENING

The Army Corps of Engineers sponsored a study of lifelines on military bases. One of the products which came out of that work was a set of checklists for screening water supply lifelines part of which is reproduced below.

Pump Stations

T F **PIPING PENETRATIONS:** Piping at wall penetrations and equipment has flexible connections or sufficient clearance.

T F **ANCHORAGE:** Pumps, motors, control cabinets, generators and controls are adequately anchored.

T F **VIBRATION ISOLATORS:** Vibration isolated pump and drive units have seismic snubbers to limit motions.

T F **OFF-SITE POWER:** Off-site electrical power or has backup provisions.

Process Tanks And Structures

T F **ANCHORAGE:** Tanks are adequately anchored.

T F **IMMERSED COMPONENTS:** Concrete or timber baffles, rotating equipment, and other immersed components have been designed for sloshing and inertial effects.

T F **PIPING PENETRATIONS**: Tanks have flexible connections at piping penetrations.

T F **LIQUEFACTION**: Structures are not buried in liquefiable soil.

Equipment and Piping

T F **ANCHORAGE:** Plant equipment is adequately anchored.

T F **COMMON FOUNDATIONS:** Pumps and motors are on common foundations.

T F **PIPING CONNECTIONS:** Flexible piping connections are used on all equipment.

T F **EXPANSION JOINTS:** Piping which crosses expansion joints has flexible connections.

T F **BRACING:** All piping runs are transversely and laterally braced.

T F **HAZARDOUS MATERIALS PIPING:** All piping runs are transversely and laterally braced. Provisions for containment are present in the event of a breach.

T F **FALLING DEBRIS:** Falling debris cannot damage yard and plant piping.

T F **HORIZONTAL TANKS:** Horizontal tanks (including fuel, liquid natural gas, propane, diesel, chemical) are adequately anchored.

T F **ELEVATED TANKS**: Elevated tank and equipment legs are adequately braced.

Pipelines

T F **BACKFILL AND BEDDING:** Pipes are buried in compacted bedding and fill.

T F **COUPLINGS**: Couplings are flexible with rubber gaskets.

T F **MATERIALS**: Pipes are constructed from appropriate materials

T F **FAULT CROSSING**: Pipes do not cross active earthquake fault zones.

T F **ELEVATED PIPES:** Elevated pipes are braced for longitudinal and transverse movements.

T F **PIPING PENETRATIONS:** Pipes have clearance and flexible couplings at wall penetrations.

T F **CORROSION:** Internal and external corrosion has been studied and does not affect seismic performance.

Storage Tanks

T F **SEISMIC SHUT-OFF:** There is an automatic earthquake-triggered shut off valve.

T F **PIPING PENETRATIONS:** Piping connections have seismic joints which allow rotation and axial movement.

T F **ANCHORAGE:** Steel tanks are anchored.

T F **ANCHOR EMBEDMENT:** Anchor bolt and strap embedment will develop yield strength.

T F **ANCHOR DUCTILITY:** Anchor bolts and straps have at least 6 inches stretch length above the foundation.

T F **WIRE WRAPPED TANKS:** Wire-wrapped concrete tank reinforcing is not corroded.

T F **SLOSHING:** Roofs and supporting columns are designed to resist the effects of sloshing water.

T F **FOUNDATIONS:** Differential settlement, liquefaction, landslides or fault rupture are not expected at this site.

T F **WELD CORROSION ALLOWANCE:** Steel tank weld thickness was increased to allow for corrosion.

T F **TANK BRACING:** Elevated tank legs are braced.

T F **COMPRESSION BRACING:** Elevated tank leg bracing has significant compression capacity.

Containment Reservoirs for Tanks

T F **LIQUEFACTION:** Earth berms will not liquefy.

T F **LINING**: The reservoir is lined.

T F **SEISMIC SHUT-OFF**: There is an automatic earthquake-triggered shut off valve.

Lifeline Support Buildings

T F **BUILDINGS:** Buildings have been evaluated and found acceptable according to FEMA procedures.

T F **EXITS**: Suspended equipment over exit corridors is has adequate lateral bracing and vertical support.

T F **EXHAUST FANS**: Failure of exhaust fans will not create areas with a hazardous atmosphere.

T F **ANCHORAGE**: Office and lab equipment is adequately anchored.

T F **COMPUTER FLOORS:** Computer floor pedestals are braced along every grid line. Pedestals and braces are bolted to the floor.

Electrical Power

T F **OFF-SITE POWER:** Failure of off-site electrical power will not affect operations.

T F **ANCHORAGE:** Transformers, control cabinets, switchgear, motor control centers, etc. are adequately anchored.

T F **POLE MOUNTED TRANSFORMERS:** Pole mounted transformers are laterally braced and anchored.

Uninterrupted Power Supply

T F **ANCHORAGE:** Charger and invertor units are anchored.

Emergency Power Engine Generators

T F **ANCHORAGE:** Generator is bolted to the floor.

T F **VIBRATION ISOLATORS:** Isolators and retainers are not cast iron.

T F **SNUBBERS:** Vibration isolators have seismic snubbers.

T F **SUPPLY LINES:** Fuel, electric, cooling water, air start, exhaust and water lines can accommodate relative movement.

T F **FUEL TANKS:** Fuel tanks are adequately braced and anchored.

T F **COMMON FOUNDATIONS:** Engines and generators are on common foundations.

T F **DIESEL FUEL:** Diesel fuel is changed at least once per year to prevent clogged fuel filters and injectors.

T F **COOLING SYSTEM**: Cooling system does not leak and has enough make-up water.

T F **SYSTEM LOADS**: System loads have not increased since the generator was installed.

T F **AIR START:** Air start system compressor and air tanks are adequately anchored.

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CHAPTER 6 ECONOMIC / RISK ANALYSIS

Introduction

The 1990's represent a period in which both government and industry are attempting to reduce expenditures, focusing on economics of operation as a priority problem. Both are undergoing a downsizing to eliminate unnecessary functions and personnel with an increased emphasis on cost effectiveness and maximization of return on investment. All construction has a purpose and the economics of use is involved in the decision process to build or upgrade. Commercial and industrial construction are categories of investment which generally are designed to serve in an income-producing role. The user commits to the expenditure of an amount of resources to establish an operating environment to meet a specific objective. In the corporate world, the objective may be an industrial complex designed to produce a product. For this application the objective of the investment is a marine oil terminal designed to serve as a means of transferring oil from a ship or barge to a shore based facility. The facility represents a costly investment to the owner/operator. It also represents a vital resource to the State of California as a means of supplying the fuel needs of the State. In addition to the economics of operation there is the additional concern of protection of the environment.

The California State Lands Commission has oversight of over sixty marine oil terminals, some of which are over eighty years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected. The criteria uses the factor α to relate the seismic exposure period of existing construction to that of new construction. In effect α is the main factor which determines the seismic upgrade level for an existing facility. The choice of what α to use is an economic decision on the part of the owner and a risk acceptance decision on the part of the State. Although there is need of definition of a minimum value of α from a regulatory perspective, the decision of what α to use should be based on maximization of benefits and minimization of risk. The CSLC goals are to:

- Ensure safe and pollution-free transfer of petroleum products between the ship and land based facilities.

- Ensure the best achievable protection of the public health, safety and the environment

- Maximize the utilization of limited resources

The development of guidelines in part involves prescription of a set of constraints to minimize the size and frequency of an oil spill. This imparts some design requirements and imposes some expenditure of money to build a system to which will not fail under some prescribed load conditions. An important issue is the degree of severity of the design requirements. This must be viewed in terms of the consequences of the resulting failure. Over the last forty years, the evaluation of risk and consequences has been advanced starting with work on nuclear power plant safety. Risk analysis and economics have been utilized in transportation both in the design of automobiles and highways. From this certain norms have evolved. Society

is much more adverse to a single catastrophic event than equivalent damage spread over a number of events, such as a plane crash versus highway deaths. The following table illustrates society's aversion to events perceived as catastrophic.

	Catastrophic	**Critical**	**Marginal**	**Negligible**
Frequent $X > 10^{-1}$	Unacceptable	Unacceptable	Unacceptable	OK
Probable $10^{-1} > X > 10^{-2}$	Unacceptable	Unacceptable	Undesirable	OK
Occasional $10^{-2} > X > 10^{-3}$	Unacceptable	Undesirable	Undesirable	OK
Remote $10^{-3} > X > 10^{-6}$	Undesirable	Undesirable	OK	OK
Improbable $10^{-6} > X$	OK	OK	OK	OK

40 CFR 300.5 (NCP) defines a major spill as in excess of 10,000 gallons (238 barrels). A consensus of persons contacted from agencies such as the Coast Guard, Minerals Management Service, and oil removal contractors indicate that in excess of 1000 barrels constitutes a large spill of potentially enormous consequences if it reaches a shoreline. Most people would say that a spill of 1200 barrels would constitute at least critical consequences. A few might say that under the most adverse circumstances, catastrophic consequences might occur. The extent of the damage depends on a number of factors including the nature of the shoreline, the composition of the oil, wind speeds and temperature etc.

1200 barrels is a large critical spill

The federal government has in some instances taken a position ignoring risk and acting as a self-insurer. This is possible chiefly because of its huge size. Other entities both state and private do not have this ability. Risk must be considered as an integral part of decision making. A prudent investor does not always seek the highest yield alone; rather one must also consider the volatility (riskiness) of the investment decision.

This chapter will introduce techniques which had there origins in the evaluation of alternatives largely based on economic issues and expands on those techniques to include risk of adverse consequences.

Cost of an Oil Spill

The cost of an oil spill involves several elements. There is the direct cleanup cost involving the expenditures on removal of the oil. There is the cost of damage to the coastline and the environment in the form of the destruction of wild life and natural resources. There are third-party damages consisting of individuals who suffered property damage from contact with the oil. Additionally there are factors such as loss of use.

The State of California Office of Oil Spill Prevention and Response estimates the cost of an oil spill based on an average of 108 oil spill incidents as follows:

Cleanup cost	$150 /gallon
Third-part cost	$100 /gallon
Natural resource damage	$200 /gallon
Total Cost	$450/gallon

Noting that there are 42 gallons per barrel, the cost of a 1200-barrel spill would be $22,680,000. The 1990 Oil Pollution Act establishes a level of financial responsibility for a 1000-barrel oil spill in federal waters at $35 million.

Potential damage from a 1200-barrel spill is very large

The costs associated with an oil spill must be factored into the decision making process for selecting the design α for a seismic upgrade.

Economic Analysis

In the 1980's the Naval Civil Engineering Laboratory, now named the Naval Facilities Engineering Service Center, developed a procedure for the economic analysis of seismic design levels and lateral force resisting systems, Ferritto (1982, 1983, 1984a and 1984b). That work led to the development of Chapter 7 of NAVFAC P355.2, Seismic Design Guidelines For Upgrading Existing Buildings. The procedures have been adopted for use by the engineering community and used to analyze the seismic upgrade of several hospitals. Recently the State of California passed SB920 which mandates an economic analysis be conducted when new earthquake hazard mitigation technology such as base isolation or viscoelastic dampers are proposed for use in State construction projects. The State of California has adopted for use the economic analysis procedures developed by the Navy referenced above. New data on damage was added. The State of California procedures for conducting an economic analysis are contained in "Earthquake Hazard Mitigation Technology Guidelines", Way (1995). This section will present the general procedure which although developed for buildings is directly applicable to any waterfront structure.

Economic analysis techniques have been used extensively in business and engineering. There has been investigation of the cost of seismic construction upgrading in a number of

documents such as FEMA 157 (1988). FEMA 228 (1992) and 229 (1992) discuss a benefits-cost model for the rehabilitation of buildings. A significant study was performed by the Applied Technology Council, ATC-13 (1985). These studies took a macro-level perspective looking at the decision process for large inventories of buildings, expressing costs on a per square foot basis, and developing guidelines for application to classes of construction. The models for estimating cost and damage focused only on evaluating the lateral force resisting system. There have been a number of studies of damageability and a good summary of this topic is found in Taylor ed. (1992). Harris and Harmon (1986) performed an economic analysis using techniques very similar to those outlined in Ferritto (1984a), but the work was unfortunately oversimplified to the point where its results are limited. They related damage only to drift and failed to include story force/acceleration as a separate damage mechanism. Ductility demand alone can not represent all damage since direct force/acceleration effects on elements mounted to floors or ceilings and damage to building contents would not be included. One would erroneously conclude that simply stiffening a building would reduce all damage when in effect we find that induced floor accelerations are increased by stiffening. One would never be able to completely assess the cost - benefits of base isolation if acceleration damage were omitted. Their damage function for the total building consisted of interpolating between yield and collapse ductility levels for only the lateral force resisting element neglecting the possibilities of different level of damage to the other building elements and subsystems.

There is an increased emphasis on post-earthquake facility functionality by the engineering community. In this light, it is essential to be able to evaluate the extent and location of expected building damage. Are there any weak links in the facility system design which will preclude operability? Operability demands that the facility be viewed as a total system not just a structural system. Utilities and the other elements must function to have operability. It is necessary to know what other facility system elements are damaged in addition to the damage to the lateral force resisting system. This section presents a detailed analysis procedure which can evaluate the economics of seismic design for a building system.

The purpose of this analysis procedure is to perform an economic comparison of alternative designs of a structure considering initial construction expenditures and expected earthquake induced damage over the life of the structure. It may compare different types of construction or different design levels. It is thus intended to assist the user and the design engineer in obtaining cost effective seismic construction. The procedure referenced above is a process of estimating earthquake damage based on both displacement and acceleration. As such it recognizes that the facility system is composed of components, some structural, some nonstructural and some mechanical and electrical, which are affected by displacement or drift. It also recognizes the damage induced in some facility system components which are mounted to floors or ceilings are damaged by the transmitted story accelerations. The procedure of including both drift and acceleration is a significant factor in this procedure which is an improvement over other techniques which focused only on drift. As noted above, failure to include the acceleration induced damage leads to erroneous conclusions that mere stiffening which reduces drift is fully effective. For every dollar that is invested in stiffening a structure, a portion of it may be wasted because stiffening results in increased floor accelerations which can cause additional damage to acceleration sensitive components like contents.

The methodology referenced above used available data at the time of its writing; since then the Loma Prieta earthquake of 1989 and the Northridge earthquake of 1994 coupled with extensive university testing have greatly increased the damage data base. In the process of developing the State of California guideline, the original damage estimation tables were updated to include the new data. This new database is now available and was used to update damage relationships, Way (1995). The procedure for conducting an economic analysis is applicable to both new and existing structures. The procedure is appropriate for larger projects which can justify a site seismicity study and the additional steps involved. The procedure is not meant for structures where the building code is design is adequate, but rather for those structures where post-earthquake performance is under consideration. It is best applied during the design process when cost estimates of the proposed structure are usually made and the performance of the structure analyzed. When only relative performance of alternatives is required, the general procedure may be shortened as will be described in following sections.

Steps for Economic Analysis

The following illustrates the steps in an economic analysis. While the procedures are illustrated in terms of a building example, they are applicable to piers and wharves and other facilities found in marine oil terminals.

Define System Components (Step 1) The system and all its component elements must be identified. This includes site location, structural plan, key facility components, utilities and lifelines. This step quantifies the operating goals and performance objectives.

Development of Alternatives and Alternative Costs(Step 2) The analysis may be applied to new construction to evaluate:

- alternative structural systems
- alternative materials,
- alternative concepts such as conventional construction vs. new earthquake hazard mitigation technology such as seismic isolation
- alternative seismic design load levels such as various design acceleration levels
- alternative earthquake ground motion recurrence intervals

For existing construction, analysis may be applied to evaluate:

- alternative seismic upgrade levels
- alternative concepts of upgrade including conventional construction vs. new earthquake hazard mitigation methods

When an analysis is applied to a design project considering alternative concepts, it is necessary to evaluate the cost of each alternative. A preliminary structural design must be performed to determine structural member sizes for each alternative. Additionally nonstructural items affected by the seismic forces must be designed to the extent that they represent significant cost factors which vary among the alternatives. Once the structure is defined a detailed cost estimate can be

completed. This is a very important step in the analysis and one which determines the level of accuracy.

As is usual practice in preparing a cost estimate, the structure should be broken down into major components and the cost of each component noted separately. The division of the facility into components is an important step since each component will be later analyzed for damage. As will be shown later, for the case of a building, it is important to separate out components which are drift sensitive from those that are force/acceleration sensitive. Equipment mounted on floors will be sensitive to the acceleration levels it receives; while, items such as vertical plumbing risers spanning between floors will be drift sensitive. Some items will fall into both categories. Where desired, a component may be subdivided into elements for a more detailed evaluation. It is required that a detailed cost estimate be compiled for each alternative being evaluated. There may significant portions of the cost estimate which do not vary among the alternatives. The amount of work involved is not as great as it might appear. Once a routine detailed cost estimate is prepared for the basic structure concept, as is standard practice, only those elements which change among alternatives need be evaluated. Use of individual components has the added benefit of showing where the damage occurs and whether there are any weak links in the system. This is especially important for systems which are expected to remain operational after an earthquake.

While the procedure is applicable to all waterfront construction, it will be illustrated by a case study of a building for which data was available. A study was performed in which a 185-foot square three-story building was designed for various steel and concrete lateral force resisting alternatives. Five lateral force-resisting alternatives were evaluated for six design acceleration levels. Figure 6-1 shows the cost increase of seismic design as a function of the design acceleration level for the various alternative lateral force-resisting systems. For this illustration, the structure was designed to be at the elastic limit at the design acceleration level to facilitate comparison. It is interesting to note that in this case, the cost of seismic strengthening is a relatively minor part of the structure's total cost.

It should be noted that in addition to the alternatives of modification of the structural design there may exist non-engineering alternative of land-use consideration (moving to a less vulnerable site), and financial and emergency response methods. In a building, use and occupancy restrictions can have significant impact on life-safety hazards. System enhancements are another possible risk reduction method (increasing the redundancy of key operational and risk-protection elements of the system)

Seismic Hazard Identification and Assessment (Step 3) Fundamental to evaluating the potential for seismic damage is quantifying of the hazard exposure. This is accomplished by a site seismicity study which determines the intensity and characteristics of ground motion shaking which pose a risk to a specific location. The method of performing a site seismicity study has become standard practice and is used by many geotechnical firms. In general, an historical epicenter database is used in conjunction with available geologic data to compute the probability distribution of site ground motion. The process of quantifying the level of hazard involves building a seismic model of the region using epicenter data, tectonics and geology. (See Chapter 2) The results of the seismicity study which are used herein include:

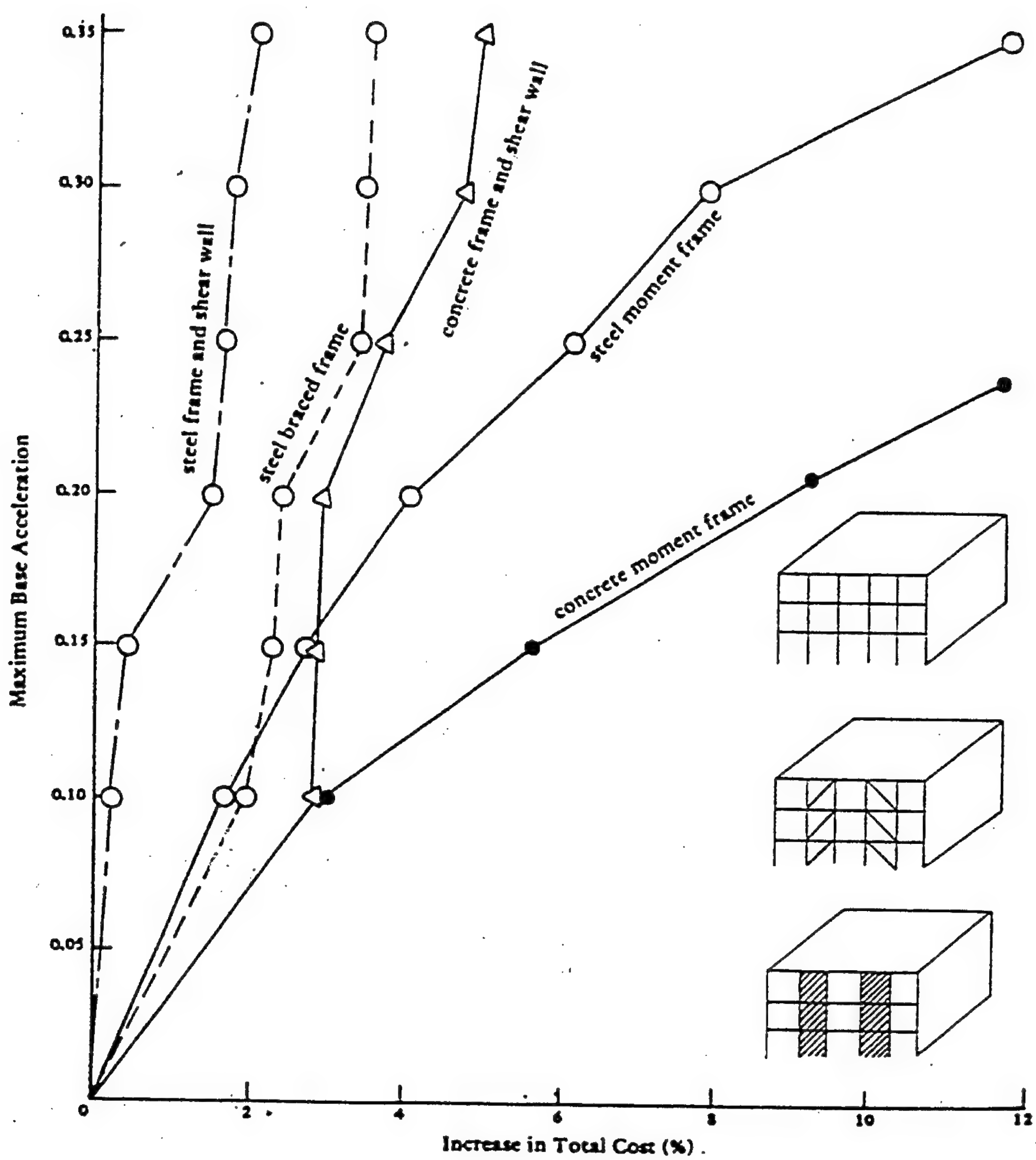


Figure 6-1. Cost of seismic resistance alternatives in new construction.

- Definition of the site acceleration probability distribution
- Definition of an array of causative (potential damage producing) events in which magnitude, separation distance, and site acceleration are defined forming a probabilistically complete set of events of significance to the facility in terms of damage causation.

Figure 6-2 illustrates a typical non-exceedance probability ground acceleration distribution for a site for a given exposure period. The word "total" is used because it represents the combined effects of all seismic source zones acting on the site. A histogram can be constructed showing the expected probabilities of various levels of ground shaking, Figure 6-3. Development of Figures 6-2 and 6-3 are the first steps in the economic analysis and are based on information available usually part of a routine seismicity study for a large facility. The use of the probability distribution and the array of discrete damaging earthquakes represent a complete set of data defining the total seismic hazard. As such it mathematically captures the exposure hazard.

Damageability Evaluation (Step 4) For waterfront construction it is necessary to consider all damage mechanisms on the structure. These include the shaking damage potential to the structures directly. They may also include other elements such as:(a) potential damage due to liquefaction and ground movement, as well as ground shaking; (b) repair cost issues for such facilities, such as possible difficulties due to lack of accessibility (e.g., to repair or replace underwater or underground piles that are damaged); (c) for major ports and marine oil terminals, the potential significance of major secondary economic losses due to interruption of operations and effects on other stakeholders; and (d) the potential for earthquake-induced environmental damage at these facilities.

Earthquake induced structural damage is caused principally by two mechanisms: drift and forces/accelerations. Drift is the mechanism usually causing damage to structural systems. There have been numerous tests conducted of lateral structural resisting systems which show the strength of these elements under cyclic load reversal. Building elements anchored to floors or suspended from ceilings feel the floor acceleration and respond as substructures. Depending upon the natural period of the structure, floor accelerations can be significantly higher than surface ground motion levels and tend to increase with height within the structure. The original Navy work, Ferritto (1984a), presented data tables relating damage of various components to drift and to acceleration. Way (1995) has updated this information based on experience over the last decade. Figure 6-4 gives the most current damage estimate data.

For each alternative it is necessary to conduct a series of analyses to compute damage over a range of possible ground motion levels. Looking at the probability histogram of occurrences of various levels of acceleration in Figure 6-2, it can be seen that the bins cover increments of 0.1 g over a range of 0 to 1.0 g for the particular site.

To illustrate the process, a set of ten dynamic analyses starting at 0.05g to 0.95g would be appropriate for this case to cover the range of possible accelerations which could produce expected damage of significance. (Note 0.95g was selected upper limit for this example and would be based on the actual site data covering the upper bound acceleration at a meaningful probability.) For a specific alternative, a basic finite element model would be constructed; then,

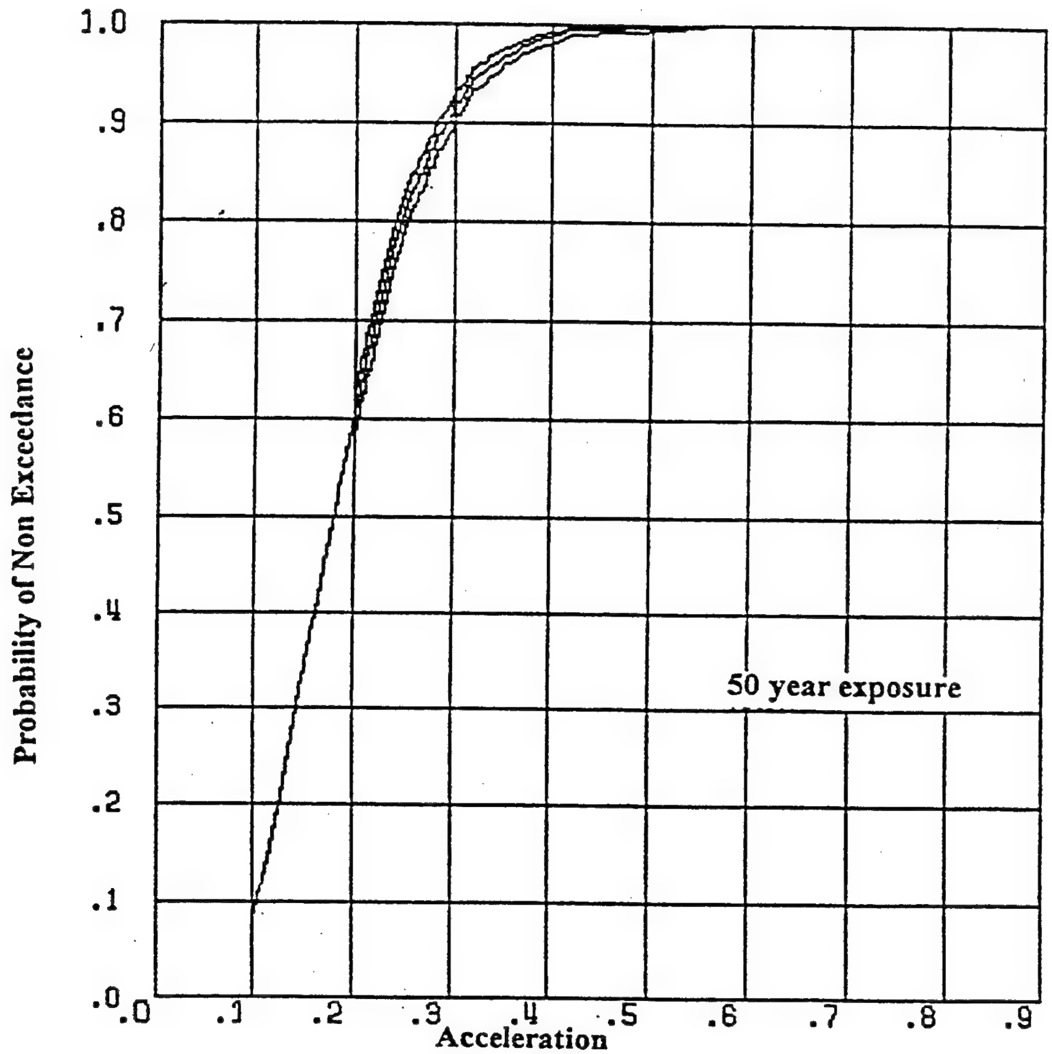


Figure 6-2. Total probability of non-exceedance of site acceleration.

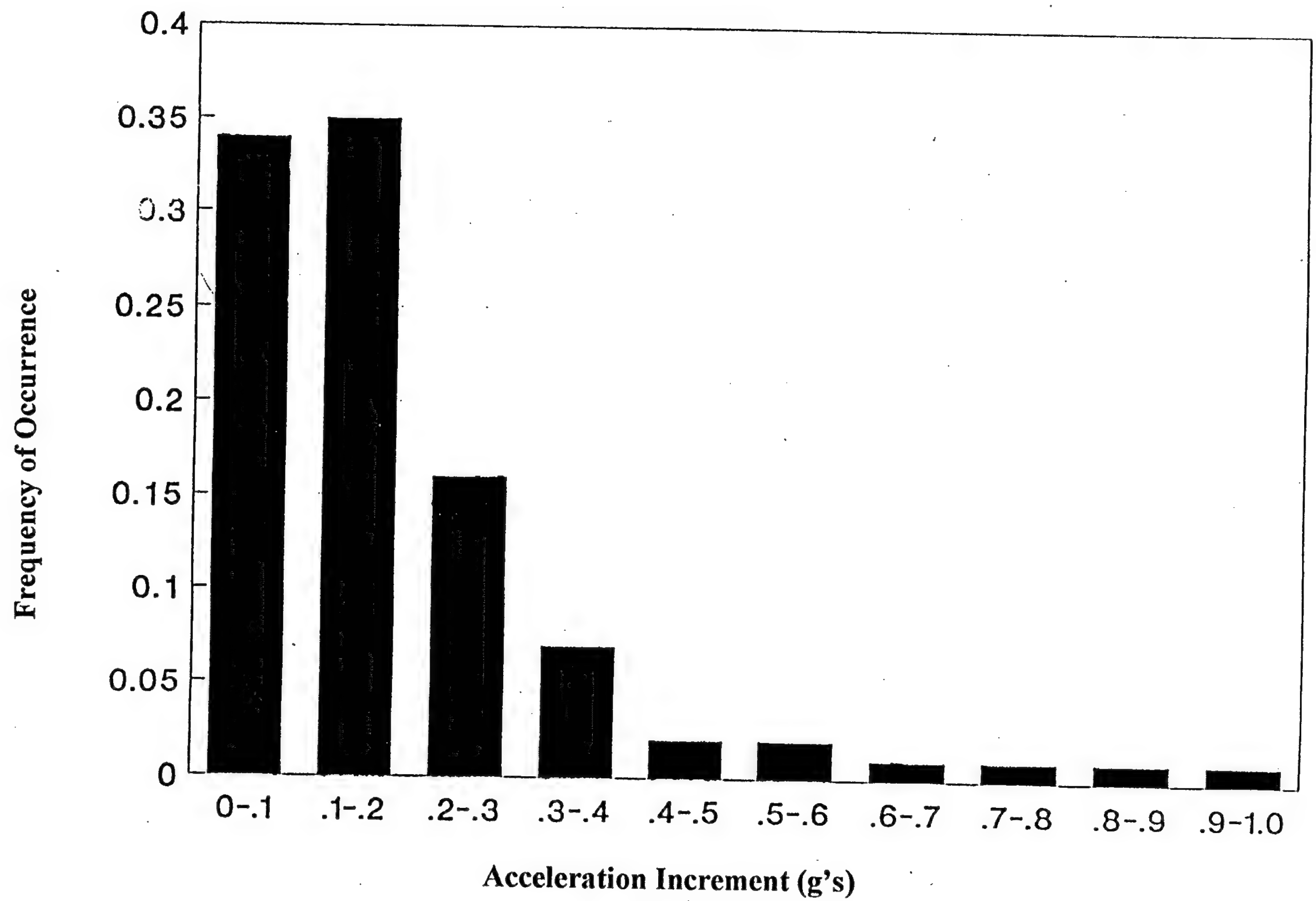
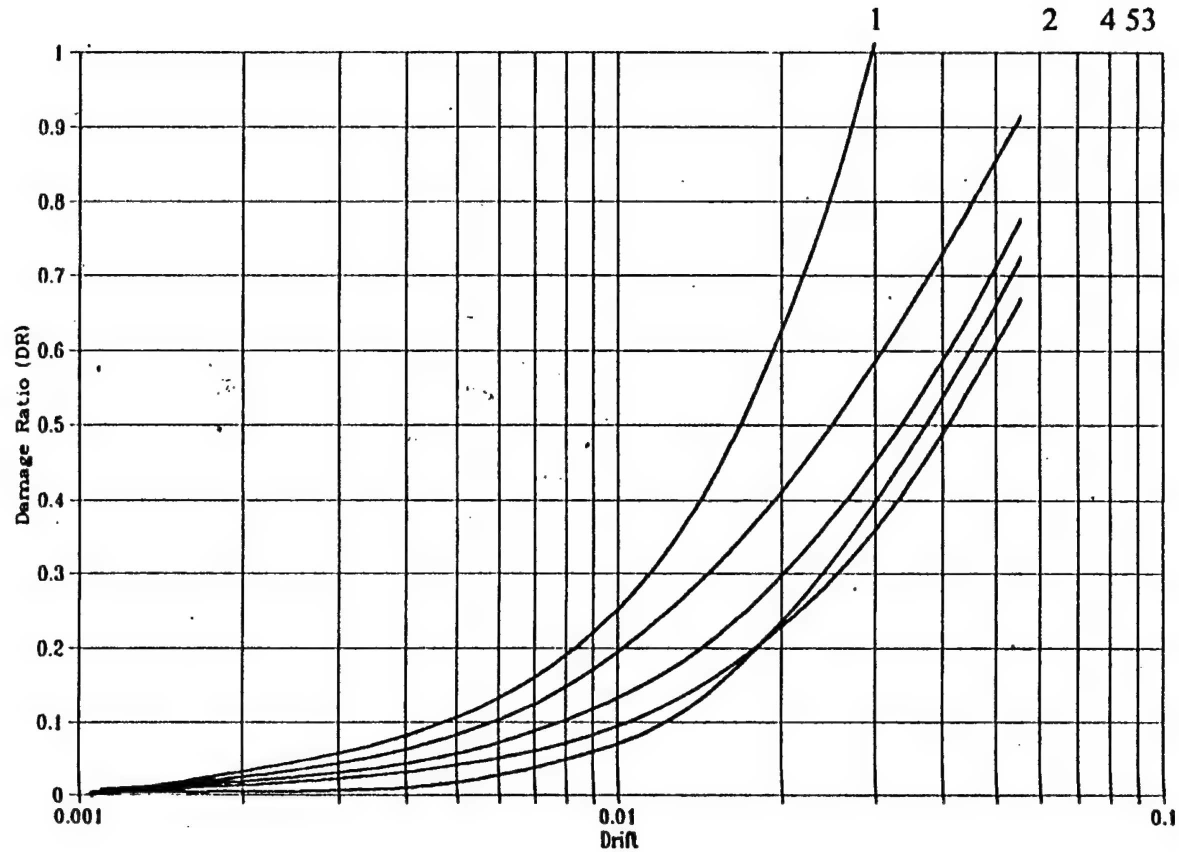
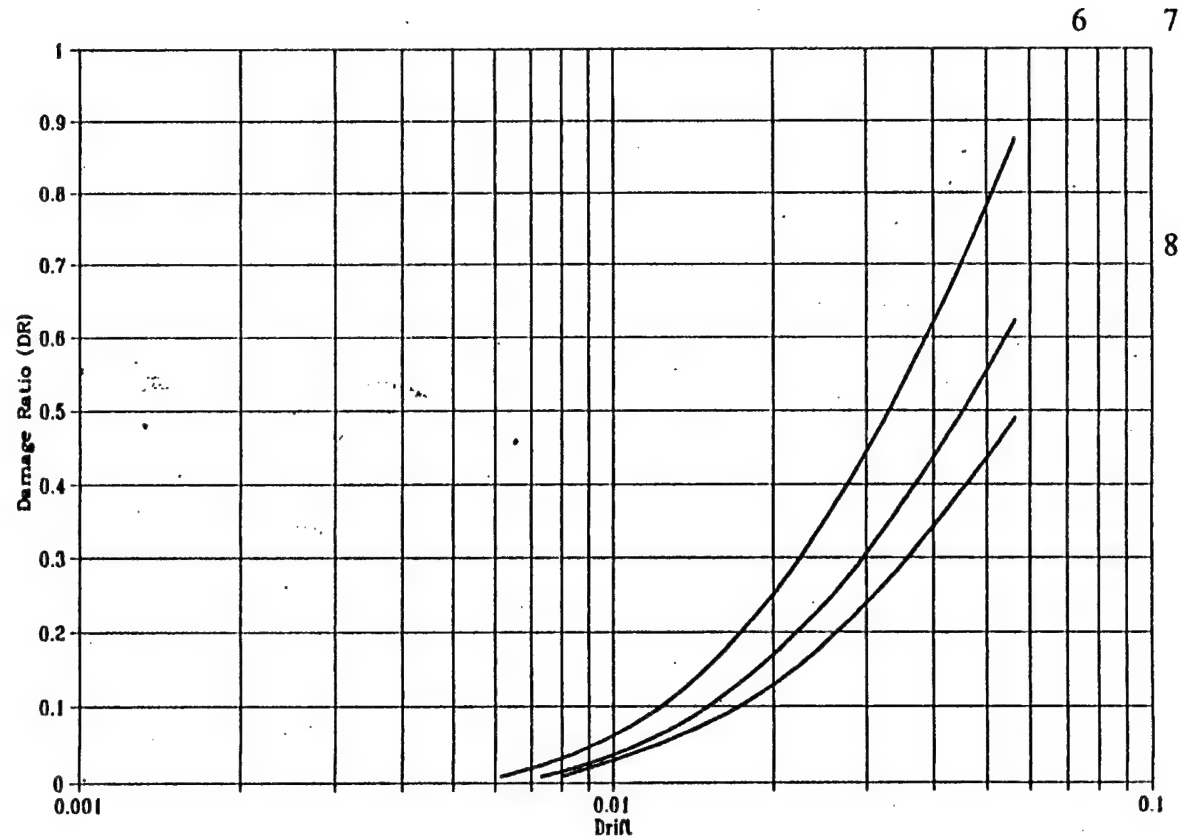


Figure 6-3.. Incremental acceleration level.



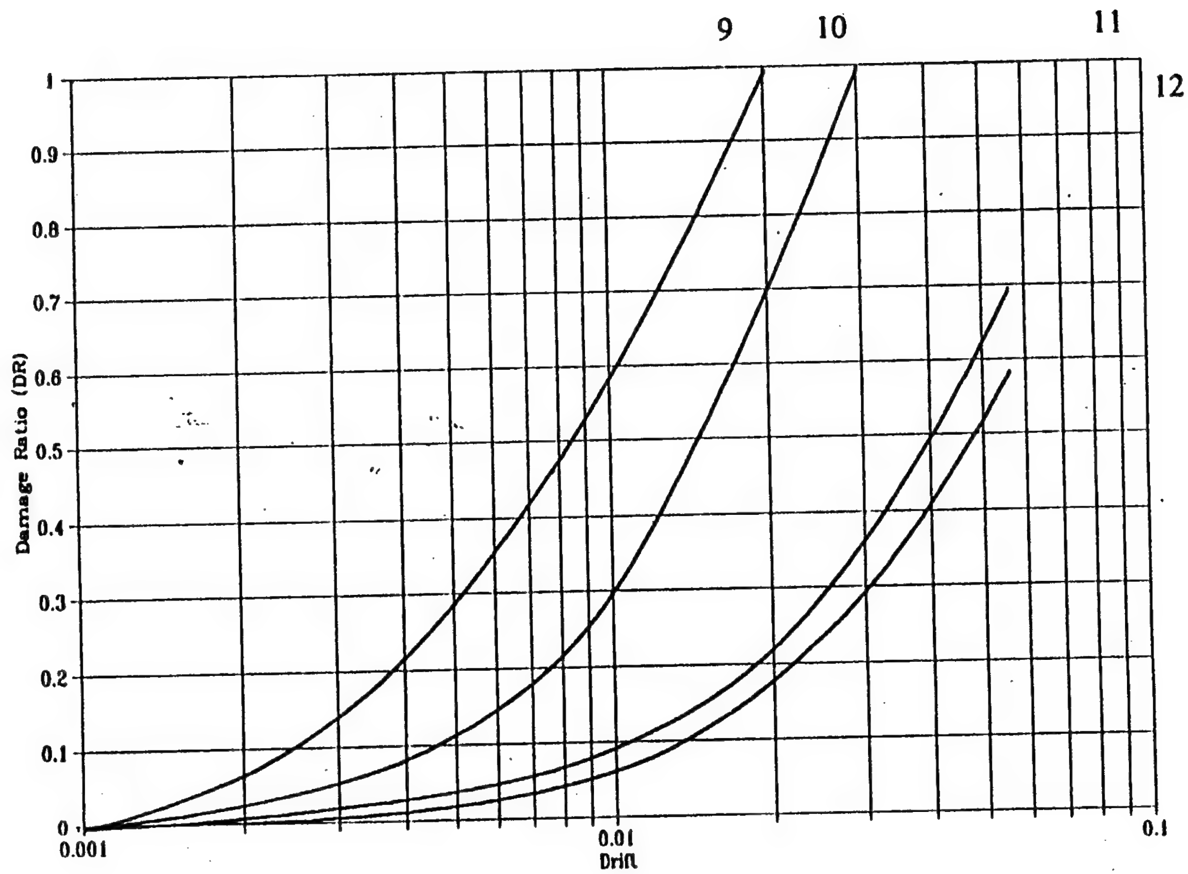
1. Masonry Walls
2. Concrete Shear Walls
3. Concrete Moment Frames
4. Steel Braced Frames
5. Steel Moment Frames

Figure 6-4. Damage as a function of drift and acceleration. (based on Way (1995))



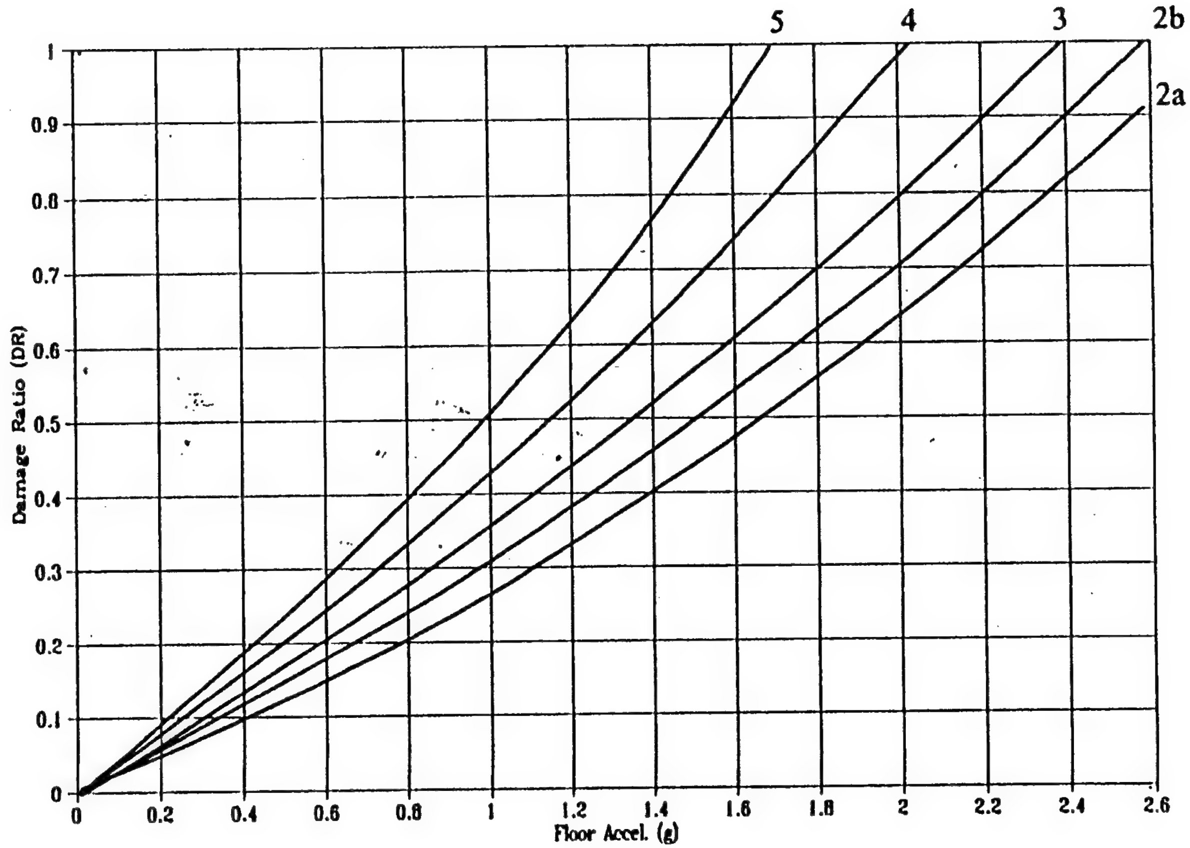
6. Structural Frames
7. Structural Floors
8. Foundations

Figure 6-4. Continued.



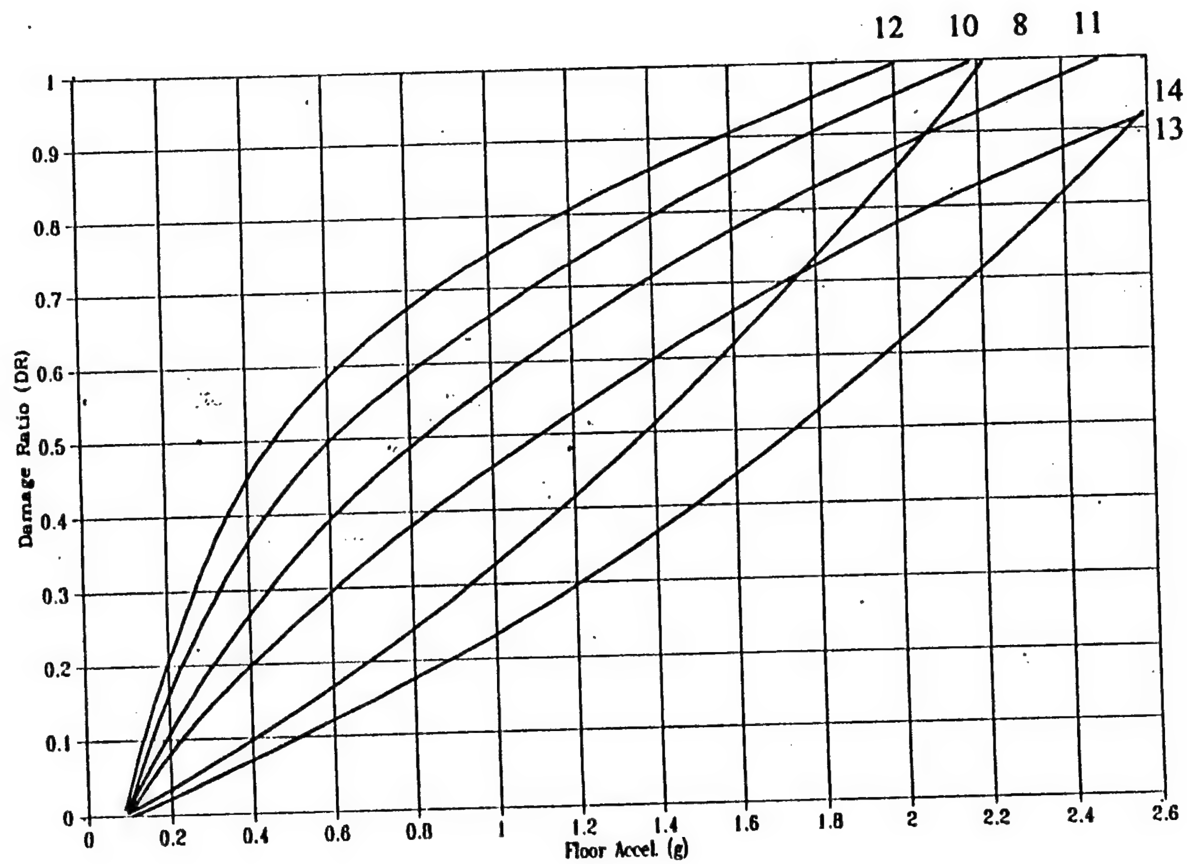
9. Architectural Glass
10. Partitions and Ceilings
11. Mechanical and Electrical
12. Contents

Figure 6-4. Continued.



5. Steel Moment Frames
4. Steel Braced Frames
2a. Steel Frames w/ CSW
3. Concrete Moment Frames
2b. Concrete Frames w/ CSW
CSW - Concrete Shear Walls

Figure 6- 4. Continued.



13. Floor Finish & Roof
10. Partitions & Ceilings
11. Mechanical & Electrical
9. Architectural Glass
14. Elevators
8. Foundation and Site Work
12. Contents

Figure 6- 4. Continued.

the ten analyses of the model would be performed in which the applied load level was increased from 0.05 g to 0.95g.[1]The results of the analysis are used to establish the interstory drifts and floor accelerations at each applied load increment. These are used to compute the damage ratio for each component by using Figure 6-4, examining the individual component elements and their appropriate drift and/or floor acceleration. The damage evaluation process is repeated for each of the ten applied load levels from 0.05g to 0.95g for each alternative. This part of the analysis can be automated by a program which post-processes the output from the finite element program and computes damage to all components and then sums component damage for overall building damage at that level of applied loading. Thus to summarize:

Alternatives 1... i

Acceleration Increments 1... j

For each dynamic analysis for a given alternative, i, and applied load level, j, each of the identified components such as structural frame, mechanical equipment etc. is evaluated for damage using the drift and floor acceleration response data. Specifically, for a typical iteration the structure is defined, the load is established, a dynamic analysis is performed, displacements and accelerations are computed (drifts, interstory displacements, floor/deck accelerations, etc), for each identified component, component damage is computed using the displacement and acceleration data, damage is summed for all components giving total damage for that iteration combination.

The element damage relationship expressed in Figure 6-4 is in terms of a damage ratio; the actual element damage cost is obtained by multiplying the damage ratio from Figure 6-4 times the element cost from the cost estimate. Alternatively the element damage can be summed to a component level based on average damage ratios and then expressed as a component damage cost based on the average damage ratio times the component cost. Experience has shown that the cost of repair is greater than the original cost because elements must first be removed before the damaged component can be repaired or replaced. A component repair multiplier, R, is used to account for this increase. The repair multipliers are based on GSA data obtained from actual experience. Note that structural materials may be in short supply after an earthquake and cost more. This may also be included in the R factor. For example, when a lateral force element is damaged, the level of damage is first computed from the drift data. This level of damage is then multiplied by 1.5 to take into account that the repair process requires more work than the initial installation. Specifically, a given level of drift may represent 10 percent damage to the element which would become 15 percent of the dollar cost of the element (10% times 1.5). The following repair multipliers are suggested to increase the component costs:

[1] The author has found that performing a nonlinear time history analyses using programs like the DRAIN2DX/DRAIN3DX computer program to be highly efficient. The amount of effort involved is not increased significantly beyond the basic analysis since repeated analyses at different load levels only involve adjusting a few parameters to change or scale the acceleration load record and the structure damping level. The topic of damping will be discussed below. No changes need be made to the structure geometry model.

Lateral force resisting system	1.5
Other structural components	1.5
Mechanical equipment	1.25
Electrical equipment	1.25
Architectural elements	1.25
Elevators	1.25
Contents	1.05

The Total Building Damage for a given iteration of acceleration load level can be expressed as:

Total Damage = Σ (Damage Ratio) * (Component Cost)* (Component Repair Multiplier)

Additional cost factors should be included in the Total Damage at this point, such as loss of life, injury and interruption in operations and lost revenue from the facility being out of service. Loss of functionality can be a very significant cost factor for certain types of facilities. The inclusion of these indirect costs are significant and can shape the results of an analysis.

The Expected Damage Cost is computed by multiplying the probability that the acceleration increment from the histogram will occur, such as Figure 6-2, times the damage or damage ratio for the building evaluated at that acceleration increment, and summed over all acceleration loading increments. The Expected Building Damage Cost for the specific alternative concept over the range of possible accelerations for the defined exposure period (for example 50 years) is given by:

Expected Damage = Σ (Total Damage for increment "bin" of acceleration) * (Acceleration "bin" Probability)

Since the damage will occur some time in the future it must be expressed in terms of the present value (PV) to relate it to the current costs of seismic strengthening or remediation.

Current Expected Damage Costs = PV(Expected Damage Cost)

In most cases, we do not have data which defines the temporal sequence of expected earthquakes over the life of the structure. It may be assumed that the risk is uniform over the exposure period. The present worth can be determined by dividing the exposure time into segments and then taking the present value of each segment.

The life cycle cost of this alternative is the sum of the initial construction cost plus the present value of the expected damage based on the preceding two equations.

Alternative Cost = Initial Construction Cost + PV(Expected Damage Costs)

Engineers have used two forms of structural dynamic analysis: response spectra procedures and time history solutions. A nonlinear time history solution is preferred because it directly computes displacements and floor accelerations taking into account structure yielding. Since there is substantial variation among earthquake records even when scaled to the same nominal peak acceleration value, the selection of an acceleration record can be a factor in establishing the maximum response of the structure. The choice of records should be examined to quantify variation in response and a series of three acceleration time histories is typically used to cover a range of response and to populate all frequency ranges of importance to the response of the structure. It is important to note that as the ratio of applied loading to design load increases, the structure undergoes increased deformation and possible nonlinear behavior. As the level of deformation increases, an increase in damping occurs which must be included in the analysis. Values for damping as a function of inelastic deformation have been discussed in the literature and are presented in Ferritto (1984a). Care must be taken at each load level iteration to select the appropriate damping for that load increment.

Decision Analysis and Alternative Selection (Step 4) At this point the owner has information which shows the cost of each alternative and the expected damage each alternative is likely to sustain over its life. The owner should examine the options in terms of the returns for investment of additional resources. Consideration of the costs of interruption of operation are essential parts of the analysis. Consideration for minimization of risk can be included and this will be further developed below.

Simple Economic Comparison - Illustrative Example

To illustrate the analysis of alternative concepts, the building discussed above will be used. The structure is a proposed three-story square building 185 feet on a side.

Problem: Consider for a new building the alternative designs of

- Steel frame and concrete shear wall
- Steel braced frame

The alternatives of frame/shear wall design and braced frame design will be compared for a 0.2g elastic design acceleration. The building is shown in plan view in Figure 6-5a and the two lateral force resisting alternatives are shown in Figure 6-5b. The components identified for analysis, their costs and repair multipliers are shown in Table 6-1. The components have been divided based on their susceptibility to drift or acceleration.

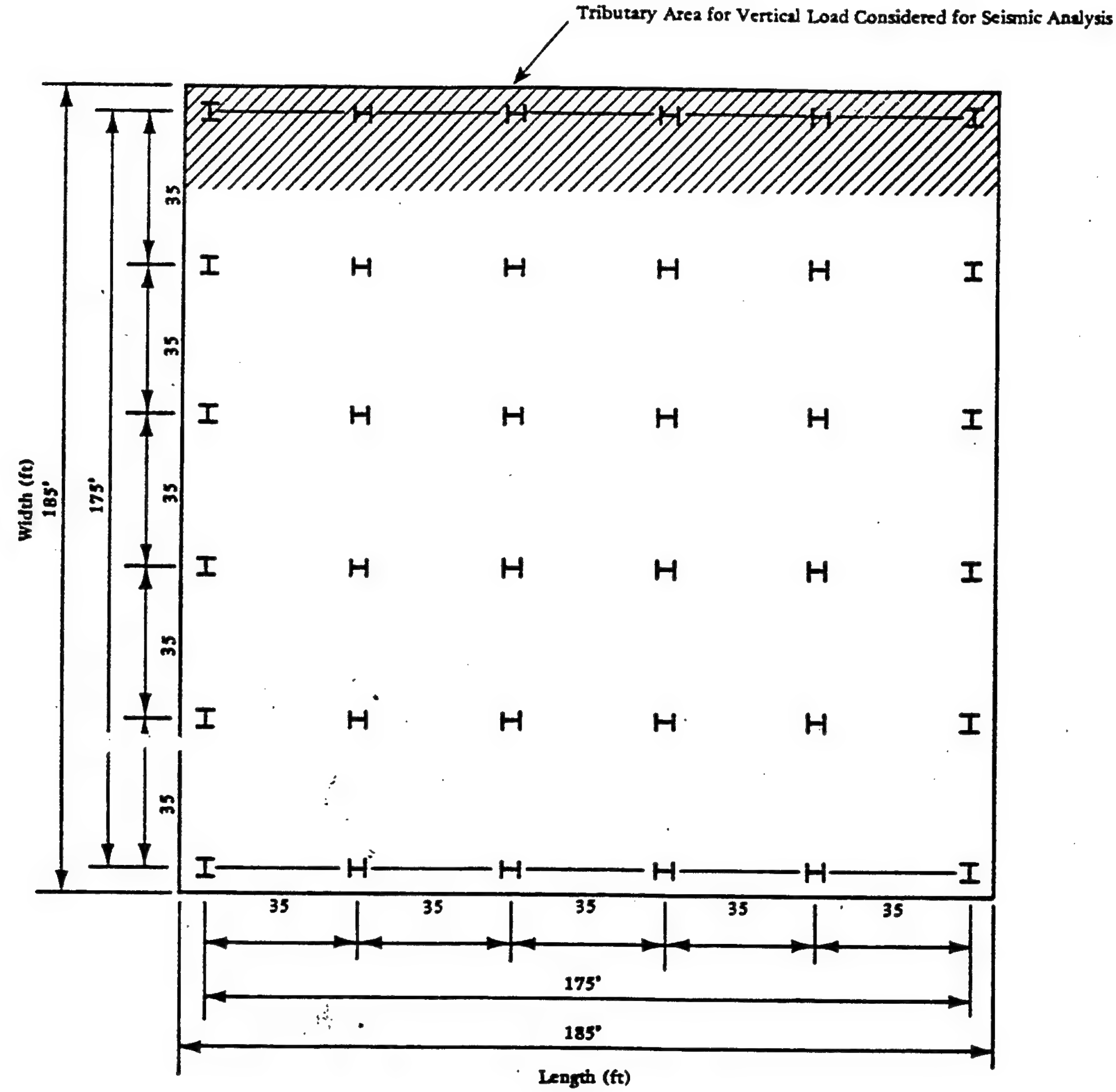
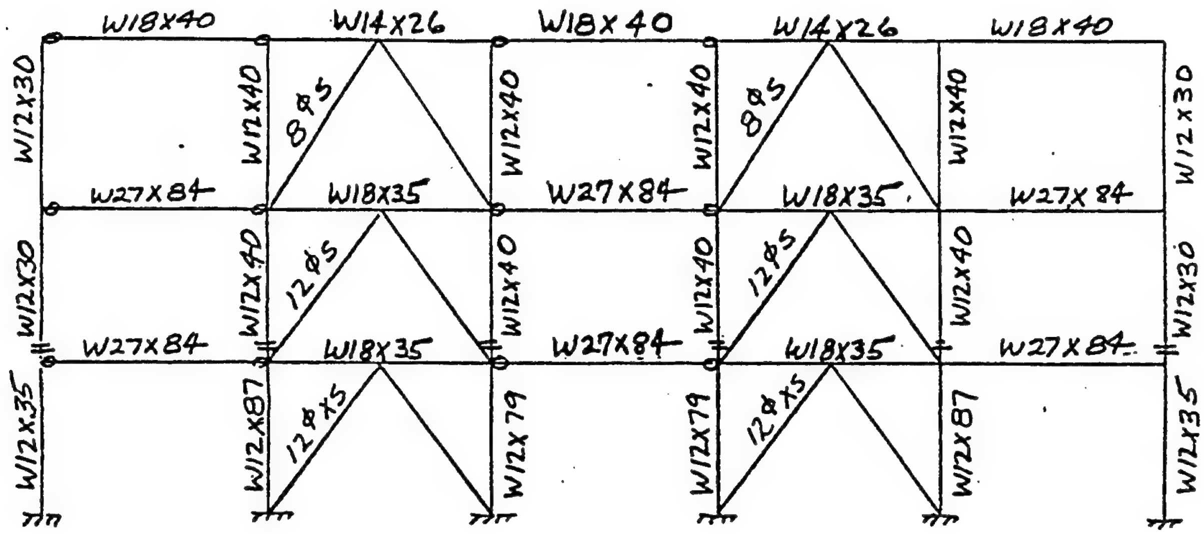
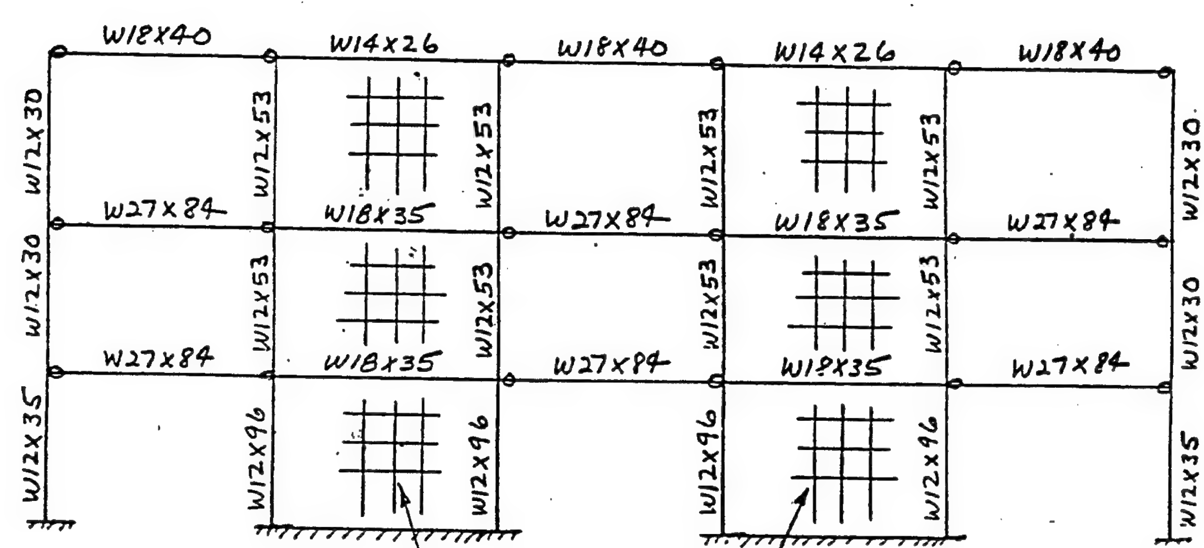


Figure 6- 5a Example building plan view.

0.20 g elastic design, $\mu = 1.0$



Braced Frame



Shear Wall

Figure 6- 5b. Sections showing braced frame and shear wall alternatives.

Table 6-1A Drift Sensitive Components

Component	Cost ($)	Repair Multiplier
1. Alternatives		
a. Braced frame	126,800	2.0
b. Shear walls	107,000	2.0
2. Nonseismic structural frame	625,500	1.5
3. Masonry	417,600	2.0
4. Windows and frames	120,600	1.5
5. Partitions, architectural elements	276,200	1.25
6. Floor	301,200	1.5
7. Foundation	412,100	1.5
8. Building equipment and plumbing	731,600	1.25
9. Contents	500,000	1.05

Table 6-1B Acceleration Sensitive Components

Component	Cost ($)	Repair Multiplier
1. Alternatives		
a. Braced frame	126,800	2.0
b. Shear walls	107,000	2.0
2. Floor and roof	301,200	1.5
3. Ceiling and lights	288,500	1.25
4. Building equipment and plumbing	731,600	1.25
5. Elevators	57,000	1.5
6. Foundation	412,100	1.5
7. Contents	500,000	1.05

The initial construction total costs for each alternative are

Steel Frame and Concrete Shear Wall $5876,700

Steel Braced Frame $5,928,800

For each increment in applied load acceleration between 0.05g and 0.95g a nonlinear analysis was performed and the interstory drift and floor accelerations determined. Specifically, the full range of accelerations which are possible to occur from 0 to maximum are covered in increments to represent a full set of motions and probabilities. The process of discretizing the acceleration loads in a set of increments does introduce some error which is believed small. Using drift and acceleration damage data from Figure 6-4, damage ratios were computed and are shown in Figure 6-6. The data in Figure 6-6 was combined with the data in Figure 6-2 to compute Total Building Damage. The calculations are shown in Table 6-2.

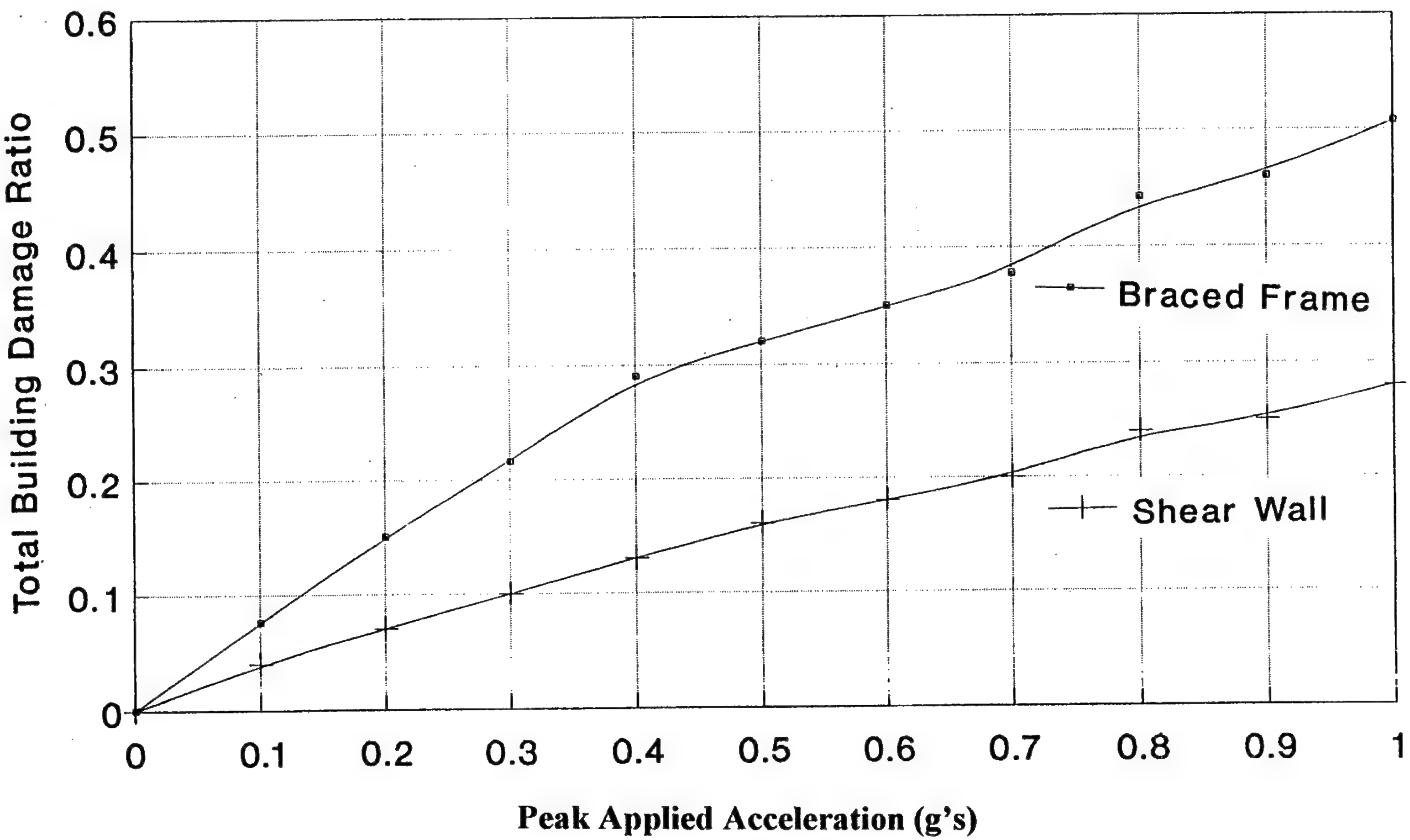


Figure 6-6. Damage ratio for alternatives.

Table 6-2. Damage Ratio and present value calculation

Acceleration Increment (g's)	(1) Frequency of Occurrence	Braced Frame (2) Damage Ratio Braced Frame	Braced Frame (1) x (2) Probable Damage Ratio	Frame & Shear Wall (3) Damage Ratio Shear Wall	Frame & Shear Wall (1) x (3) Probable Damage Ratio
0-.1	0.34	0.03	0.0102	0.015	0.0051
.1-.2	0.35	0.11	0.0385	0.05	0.0175
.2-.3	0.16	0.175	0.028	0.08	0.0128
.3-.4	0.07	0.25	0.0175	0.11	0.0077
.4-.5	0.02	0.305	0.0061	0.14	0.0028
.5-.6	0.02	0.335	0.0067	0.17	0.0034
.6-.7	0.01	0.365	0.00365	0.19	0.0019
.7-.8	0.01	0.41	0.0041	0.22	0.0022
.8-.9	0.01	0.45	0.0045	0.24	0.0024
.9-1.0	0.01	0.485	0.00485	0.26	0.0026
Total Damage Ratio		BF =	0.1241	SW =	0.0584

For 50 years of equal exposure and 7 percent interest the average Present Worth factor is 0.28 (Note this value is computed by summation of PW increments over exposure or by a random simulation)

The present value of the damage costs are:

Braced Frame 0.28 * 0.1241 * $ 5,928,800 = $206,000

Shear Wall 0.28 * 0.0584 * $ 5,876,700 = $96,000

The present worth of the future damage which can occur any time in the 50-year exposure period is determined based on the average present worth factor for increments of time using a 7 percent interest rate. Note that the 0.28 used above is the present worth of a single random damage events which can occur any time in a uniform manner in the 50-year exposure. It is computed by a Monte Carlo simulation of 1000 events. As such it represents the present worth of all future damage expressed as a single event occurring in the future and brought back to today. This assumes that all damage producing events occur at some unknown set of times in the future, that they can be summed together, and that the sum can be expressed as a single time event. The interest rate was based on the approximate rate of return on long term federal bonds and is thought appropriate for federal construction. The expected damage is:

Steel Frame and Concrete Shear Wall $206,000

Steel Braced Frame $96,000

The loss of building function from an earthquake can be a significant factor and can be included at this point. Here the user develops a value for the operation of the building in terms of the value of the product produced in the building. For administrative buildings the value of the salaries paid to the occupants can be an approximate indication of the value of the operation. As an illustration consider that the out of service lost time might be estimated as follows based on the dollar value of the damage and the time to repair:

Steel Frame and Concrete Shear Wall 10 weeks

Steel Braced Frame 5 weeks

If the building housed 200 people with a total annual payroll of $10 million, one week of lost productivity would be about $200,000 times the present value factor 0.28 or $56,000. Note this is a trivial illustration relating total lost time to total damage. It should be obvious that more complex characterizations of downtime and loss of service can and should be made based on the actual circumstances.

The total cost of the two alternatives involves summing the initial construction costs plus the present worth of the total damage and lost time costs expected. In this example they are:

Steel Frame and Concrete Shear Wall $5,876,700 + $206,000 + $560,000 = $6,642,700

Steel Braced Frame $5,928,800 + $96,000, + $280,000 = $6,304,800

Up to this point the interest rate and the life of the structure have not been discussed. Both of these can affect the choice of options. It is up to the owner/user to select these values based on the value of money to him/her and the projected useful life of the structure. For federal construction the value of borrowed money such as long term Treasury Bonds is a good indication of what money is costing. One may choose to subtract the inflation rate from the long term

Treasury Bond rate to exclude the inflation or one may add an inflation rate to future repair costs. The example assumed a constant value analysis excluding inflation. Increasing the value of the interest rate makes the present value of future losses less and reduces the economic worth of damage prevention over initial savings. It becomes harder to justify seismic damage reduction technology. Conversely if borrowed money were without cost, seismic improvements would be very attractive. Buildings tend to remain in service for long periods of time. Fifty years has been used as the economic life for federal construction. Increasing the life of the structure increases its exposure to damage but also increases the time factor in present value calculations which reduces the present worth of future damage. The specifics of the problem determine the net effect. In general the life of the structure has less effect than the interest rate.

At this point the decision-maker can evaluate the reduction in losses with increased investment. Once the minimum required standard is met, the owner may decide how much additional investment is prudent based purely on commercial business investment practice. However this may not be enough when evaluating a marine oil terminal. The risk of major spills is an important factor which must be considered and will be addressed in following sections.

Simplification of General Economic Analysis

The above procedure involves three main steps: the quantification of the seismic hazard in probabilistic terms, the determination of the initial costs of seismic strengthening or remediation, and the determination of the expected damage. It was proposed to use an incremental approach in which the ground motion acceleration probability distribution is expressed as a histogram composed of incremental "bins" of acceleration and their associated probabilities of occurrence. This produces a full and complete analysis of the best estimate of the seismic exposure. However, a site seismicity study may not always be available. The engineer is free to substitute a set of earthquake events of design interest. This set is not a complete risk assessment but rather is a comparison of the proposed structural design alternatives under an assigned set of design load conditions. Having done this, the designer may choose to consider the average performance of the structure under the assigned set of events, or perhaps the worst case event, or perhaps the cumulative effect of all the events. Again it is important to note that this approach is not a total risk analysis but only a relative comparative performance of the alternatives under a set of design conditions. It was suggested that nonlinear time history finite element models of the structure be used to estimate drift and floor accelerations using sets of time histories. The engineer may substitute elastic response spectra techniques if he chooses as long as the results are adjusted for yielding.

Application Simple Economic Analysis To Piers and Wharves

As noted above the general procedure described above for performing an analysis of design alternatives may be applied to any type of structure. Data from a recent project is available to give an indication of the cost of a pier and its components. For a 120-foot wide 1460-foot long pier to be built in San Diego, a cost of $53 million was estimated. The following gives a breakdown of elements and their costs:

Pier structure including pile foundation	\$18.1 M
Utilities	\$11.5 M
Fendering	\$ 3.1 M
Dredging	\$12.3 M
Demolition previous structure	\$ 2.6 M
Contingency etc.	\$ 5.4 M

As can be seen the actual structural costs are only 34 percent of the total project costs. Changing the pile design may influence the structure cost 15 percent but would only influence the total cost by about 5 percent.

Since the potential for harm to the environment from a large oil spill is high, the following section will address the element of risk as well as economics. The Port of Los Angeles commissioned a study of the design of the Pier 300 wharf Taylor and Werner, (1993). This study gives a valuable insight on the economics of the decision process and also allows for the inclusion of risk. The following section builds and expands on that work.

Expanded Economic Analysis To Include Risk

The preceding section presented procedures to utilize economic analysis as an aid in decision making and selection of the best alternative design. This section expands the general economic analysis procedure to include risk. The operation of a marine oil terminal is an economic process requiring prudent decision making based on business conditions and competition. However, it goes beyond simple economics because it considers potential risks to the surrounding environment due to earthquake damage to the terminal.

Overview of Procedure

This section describes an "acceptable seismic risk" evaluation procedure that can be used to provide information to enable regulatory agencies, owners, and other marine oil terminal stakeholders to make rational decisions for reducing seismic risks at such terminals. This procedure is based on the premise that it is not possible to achieve zero seismic risk; that is, no matter what degree of seismic design or strengthening is implemented, there will always be some finite residual risk of unacceptable seismic performance (which may be measured in terms of release of hazardous materials, repair costs, loss of operations, etc.). The acceptable risk procedure uses state-of-the-practice geoscience, engineering, systems, and economic analysis methods to establish that level of residual risk that is "acceptable" – i.e., for which the costs required to further reduce these residual risks are so high as to be no longer acceptable. These costs may not only be economic, but may also entail other types of costs as well (e.g., the social, political, and legal costs that may be associated with a given degree of earthquake damage).

Steps

This section outlines the seven steps (see Figure 6-7) that comprise the acceptable risk evaluation procedure. The procedure may be applied to ports, marine oil terminals, or any other

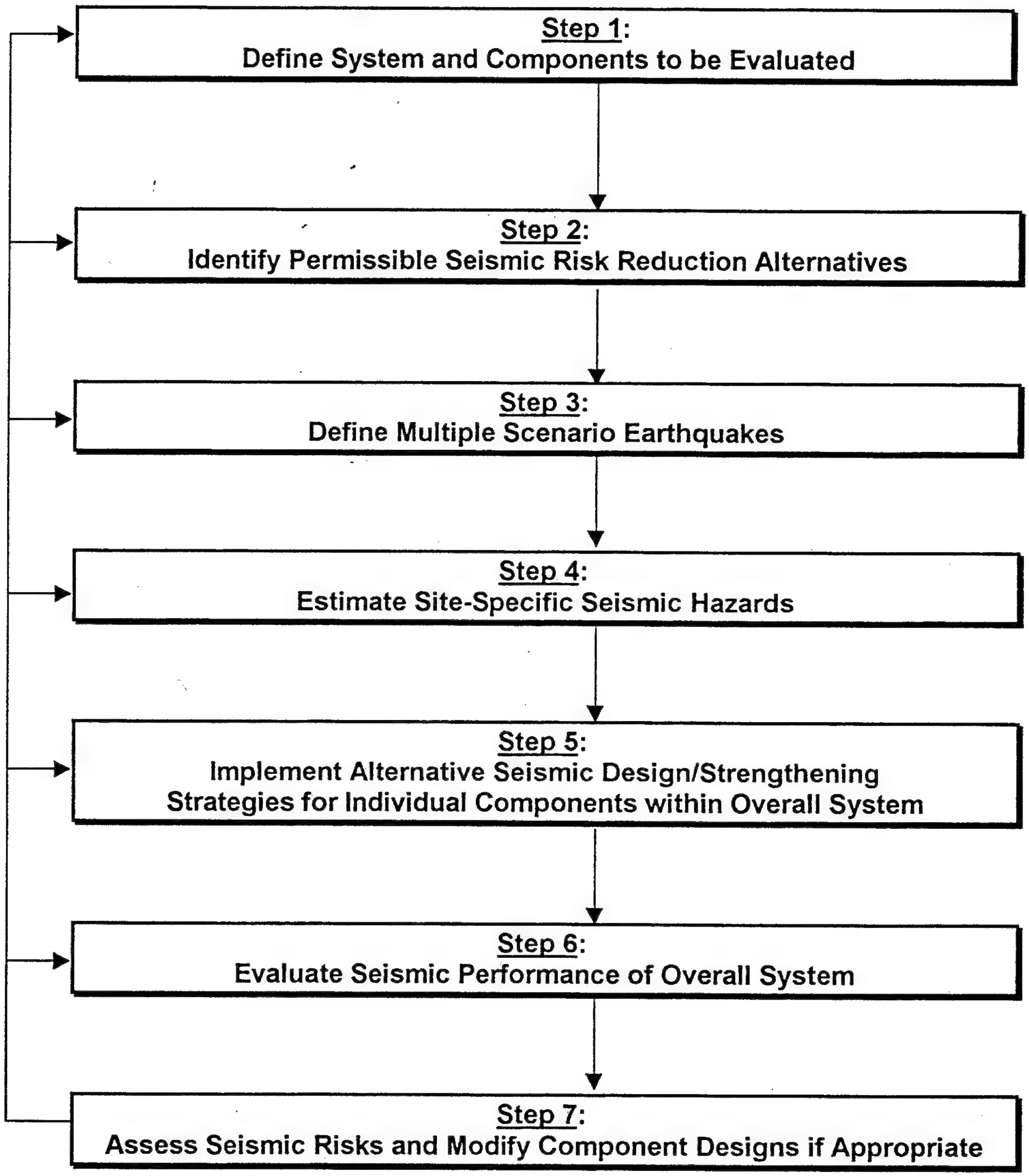


Figure 6-7
Acceptable Risk Procedure

system that may be at risk during an earthquake. As applied to different system types, the acceptable risk procedure evaluates costs and risks associated with alternative seismic risk reduction strategies.

The risks to be considered in a given application may differ according to the system type and objectives of the decision-makers. For example, decision-makers at a large commercial container port (e.g., port owners and tenants) may focus their evaluation on the reduction of risk from economic losses due to excessive earthquake damage and loss of operations. Decision makers at government regulatory agencies charged with developing performance requirements at facilities with hazardous materials (e.g., marine oil terminals) may focus on reducing risks from unacceptable release of these materials during an earthquake.

Step 1: Define System and Components to be Evaluated Under Step 1, the overall system to be considered in this evaluation is defined and described. This description should include the system's location, overall configuration, scheduled modifications, operational requirements, volumes and types of cargoes handled, and its components and their operational interfaces. The description of each component in the system should include: (a) its location(s) within the system; (b) function; (c) importance to system operations; (d) replacement costs; (e) structural elements (materials of construction, mass, e.g. location, stiffness, support conditions, etc.); (f) equipment essential to system and component operations; and (g) any prior seismic design or strengthening.

In addition, a set of operational goals should be established either for new construction or existing construction. For existing construction, the shortfalls of the present construction should be identified.

Step 2: Identify Permissible Seismic Risk Reduction Alternatives Step 2 of the procedure identifies those seismic risk reduction alternatives that are in the decision-maker's jurisdiction to implement. In general, these alternatives may include:

- *Engineering* – These alternatives most commonly consist of seismic design of new facilities, seismic retrofit of existing facilities, and improvement of potentially liquefiable soils. Engineering evaluation may also result in other measures to reduce risks such as alternative site location, occupancy reduction of less safe buildings, and use of temporary shoring.

- *System Enhancement* – The objective of these alternatives is to assure that systemic goals of the port or marine oil terminal are achieved such as maintaining cargo handling, transport, and storage operations, implementation of emergency response and recovery operations, etc. System enhancement alternatives include the development of multiple redundant operational paths and nodes for maintaining system operations and emergency power, communication, and fire fighting capability.

- *Financial Reserving* – These alternatives include the retaining of funds for emergency response and recovery contingencies.

- *Disaster Recovery and Restoration* – These alternatives include the development of post-earthquake emergency response procedures for port or marine oil terminal personnel,

stockpiling of materials and equipment, and coordination with the government, police department, fire fighting agencies, hazardous material cleanup agencies, medical agencies, and utilities.

- *Risk/Liability Transfer* – These alternatives include the use of insurance or other liability transfer mechanisms to limit post-disaster liabilities and assure that adequate recovery funds exist.

The acceptable risk procedure as described in the remainder of this section and in the example application below specifically addresses only one of these alternatives – engineering. However, it should be recognized that engineering is only one of several risk reduction alternatives that may be implemented. A comprehensive seismic risk reduction plan should encompass many or all of the various alternatives listed above.

Step 3: Define Multiple Scenario Earthquakes This acceptable risk approach applies a multi-scenario method within the framework of a Monte Carlo approach, in order to assess system costs and risks. *Scenarios* are defined as a suite of earthquakes that collectively represent the seismicity, geology, and tectonics of surrounding region. Each scenario earthquake is defined in terms of its moment magnitude and location (i.e., the location of the earthquake's epicenter, focus, center of energy release, or fault rupture zones). Only scenario earthquakes with a potential for damaging the system are considered (e.g., earthquakes with moment magnitude ≥ 5.0 and that lead to ground shaking at the site that exceeds some designated damage threshold level). The example described in the following section summarizes a state-of-the-art procedure for establishing scenario earthquakes in California.

There are many ways to develop a suite of scenario earthquakes, and to incorporate the multitude of uncertainties inherent in estimating potential future earthquakes and their locations. A Monte Carlo approach to the development of scenarios permits the incorporation of various uncertainties into the process of defining scenarios. Scenarios may be modeled in terms of one or more *simulations*. Each simulation represents the application of a random process to the independent parameters. As a consequence, to the extent that the random parameters can be modeled in terms of uncertainty distributions, a Monte Carlo approach can incorporate uncertainties in the process for selecting scenarios and the various simulations generated from these scenarios. In addition, this application of scenarios and simulations can readily incorporate spatially extended systems, such as those combining analysis of the port or marine oil terminal facility and the inland transportation systems that serve it.

To assess costs and risks over time that may be associated with alternative seismic risk reduction decisions, the scenario earthquakes may be represented in a form for use in a walk-through analysis. This form would consist of a table whose first column contains a year number (1,2, 3,….up to possibly thousands of years), and whose subsequent columns list the magnitude and location of each earthquake determined to have occurred in the region during that year. The number of potentially damaging earthquakes during each year would range from zero (during many of the years) to some maximum number, probably about 4 for California as a whole, with a smaller expected number for a facility within a specific region of the state.

Step 4: Estimate Site-Specific Seismic Hazards Under Step 4, geoscience and engineering procedures are used to estimate the seismic hazards throughout the system due to each scenario earthquake from Step 3. Strong ground motion estimates are developed for each scenario earthquake from Step 3 both as a means to estimate strong ground motion hazards and to estimate those secondary hazards such as liquefaction, slope instability, and tsunamis that may result from strong ground motions. Local fault displacement hazards are also estimated for those earthquake scenarios associated with fault systems traversing the port system in question.

Step 5: Implement Alternative Seismic Design/Strengthening Strategies for Individual Components within Overall System Under Step 5, preliminary seismic designs are carried out for all new components, for strengthening of all existing components, for ground improvement, etc. A series of alternative designs may be carried out for each component (e.g., designing each component to alternative design criteria, considering alternative seismic detailing of structural elements, alternative levels of ground improvement, alternative equipment designs and/or support systems, etc.). These designs should be taken far enough so that initial construction costs can be evaluated under this step, and overall system seismic performance can be evaluated under Step 6.

Step 6: Evaluate Seismic Performance of Overall System Step 6 provides a model of the overall system as a function of damage to each of its components. The overall system model will include (a) physical interaction effects among diverse components within the system (e.g., how damage to one component affects performance of another component); (b) direct revenue losses to the port as a consequence of damages to components and the system; and (c) impacts on other stakeholders (e.g., shippers, those living and working in close proximity to the port) of primary and secondary damage to the port.

Step 7: Assess Seismic Risks and Modify Component Designs if Appropriate Step 7 contains the following substeps that are described below.

Substep 7-1: Develop Risk and Decision Calculations for Risk Reduction Alternatives

Substep 7-1 evaluates the risk reduction alternatives from Step 2 in terms of the loss and risk estimates developed under Steps 3 through 6. These alternatives can be compared in terms of significant performance criteria. For commercial container port facilities that handle container cargo with minimal environmental risk, the performance criteria will often focus on minimizing economic risks – i.e., the potential risks of significant repair costs, business interruption losses, and higher order economic impacts due to earthquake damage (see above). For marine oil terminals that transport and store environmentally sensitive materials, these criteria will focus on minimizing environmental risks (e.g., oil spills) as well as economic risks. As discussed above, seismic risk analysis of marine oil terminals can compare the likelihood of diverse extents of oil spills to the life-cycle costs of various design and/or seismic retrofit alternatives.

An important element of this substep is the estimation of economic risks in accordance with the following considerations:

- *General.* Regardless of whether commercial ports or marine oil terminals are considered, the evaluation of economic risks associated with alternative seismic risk reduction decisions will be an important element of the risk analysis. These economic criteria used in the analysis emphasize both the mean and the variance of the life-cycle costs. Life cycle costs consider both the initial outlays (e,g., initial construction costs) and the present value of the downstream costs of alternative decisions (e.g., as noted above, the repair costs, business interruption losses, and higher order economic impacts due to earthquake damage).

- *Mean Value of Life-Cycle Costs.* Emphasizing the mean value of the life-cycle costs is best represented by a least-cost analysis. Such analysis can indicate which of the various seismic risk reduction alternatives lead to the lowest value of the life cycle costs. From an investment perspective, this is analogous to obtaining the best possible "yield" from an "investment" in seismic risk reduction. That is, if one ignores the variance of life-cycle costs, the optimal seismic risk reduction alternative will have the least mean life-cycle cost. To obtain such information, there are several reasons why a least-cost analysis is superior to a benefit-cost analysis. For example, a seismic risk reduction alternative with a ***"favorable"*** benefit-cost ratio may nevertheless not have the ***most favorable*** benefit-cost ratio. Also, some decisions, especially those involving insurance purchase, do not (or in principle should not) have a favorable benefit-cost ratio. Instead, insurance purchases are made in order to reduce the volatility of decisions.

- *Variance of Life-Cycle Costs.* Emphasizing the variance of life-cycle costs incorporates this "insurance" feature of investments. The variance represents the volatility (riskiness) associated with a given seismic risk reduction decision. In traditional capital markets, volatility is typically assessed in terms of the variance on the investment return. This is particularly important to ports, since port investments are not fully diversified, and ports do not have unlimited capital to cover investments that go bad (or are unlucky). Therefore, port investments consider volatility as well as expected value (mean) of the return on investments. These investments are primarily designed to reduce the volatility of port investments generally, and so act in significant ways as substitutes for insurance. (See Bernstein, 1996; Taylor and Werner, 1995, 1998).

- *Applicability in Acceptable Risk Methodology.* Incorporating considerations of volatility into investments is very important for natural and environmental hazards mitigation programs. It is analogous to a prudent investor who not only considers the maximum yield of an investment, but also considers the volatility of the investment. Within the context of the acceptable risk methodology, consideration of the variance of life-cycle costs is a measure of the extent to which life cycle costs due to a given scenario earthquake can deviate from the mean value computed by a least-cost analysis. Therefore this should be an important element of the seismic risk reduction decision process.

- *Discount Rate Considerations.* The application of a discount rate is necessary in economic analyses in order to compare present costs and benefits with downstream costs and benefits. However, selection of a suitable discount rate has raised many issues. Very often, the (real or constant dollar) discount rate selected is the difference between the rate for an extremely

secure (non-volatile) investment and inflation. For instance, one may select long-term federal treasury bonds as extremely secure investments, and subtract from the current rate of these long-term financial instruments the rate of inflation. Cost of capital to a port, though, may imply a slightly higher rate, since the cost of borrowing for the port may be higher than the current rate of a very secure investment.

- *Discount Rate Multiplier*. For the application of a discount rate j over an exposure time T in least-cost analysis, one may apply the following multiplier, $R_{j,T}$, to the average annualized loss:

$$R_{j,T} = \frac{1}{(1+j)} + \frac{1}{(1+j)^2} + \frac{1}{(1+j)^3} + \dots\dots\dots + \frac{1}{(1+j)^T}$$

or

$$R_{j,T} = \frac{1-(1+j)^{-T}}{j}$$

- *Applicability to Non-Economic Risks.* The application of discount rates to lives saved, injuries averted, environmental damage, and treasures lost, to mention a few categories, has raised serious questions. Is one life saved today equivalent to five lives saved in twenty years or to twenty-five lives saved in forty years? Other than in calculating the economics of health programs, can one properly discount lives saved?

Substep 7-2: Select Risk Reduction Alternative(s) that Best Fit Performance Criteria.

Under Substep 7-2, the results from Substep 7-1 are used to eliminate various alternatives and select among those alternatives that remain. For example, alternatives may be ruled out if they lead to consequences that are proscribed by regulation, or if there are some clearly superior alternatives in terms of existing performance criteria (e.g., oil spill size probabilities and total life-cycle economic costs).

Substep 7-3: Review Selections of Risk Reduction Alternative(s) with Public.

Substep 7-3 provides justification of the acceptability of the selected risk reduction alternatives through programs that incorporate public review and criticism. Stakeholders in the decision are brought in through this substep. Based on feedback from this process, one or more of the prior steps of the acceptable risk procedure, and the resulting selection of a seismic risk reduction alternative, may be revisited or modified.

Demonstration Application

Background This section describes a demonstration application of the foregoing procedure to a hypothetical container wharf at a major commercial port. In this application, costs and economic risks associated with the use of alternative design acceleration levels are compared. Information of this type provides a port decision-maker with information for making a rational decision

regarding an appropriate level of design acceleration to use for his or her wharf facility. This demonstration application is designed to be tractable, in the sense that other investigators should be able to replicate the results (except, perhaps, for the numbers resulting from application of random generators). The text of this section contains example calculations to assist in this replication.

This application is a modification of an analysis previously carried out for the Port of Los Angeles (POLA) and described elsewhere (e.g., Taylor and Werner, 1995 and 1998; Werner, Dickenson, and Taylor, 1997; Werner, Thiessen, and Ferritto, 1998). In view of these modifications, this example does not directly reflect the details and conclusions of the prior work. The main difference between the current example and the previous analysis is that the current example uses a much more complete scenario earthquake representation for the region. This current representation contains over 13,000 scenario earthquakes that cause peak ground accelerations at the site in excess of 0.01 g. As discussed subsequently, this representation was developed by adapting earthquake modeling procedures for California that were developed under the USGS National Hazards Mapping Program (Frankel et al., 1996). In the previous example, only 24 scenario earthquakes were considered that were based on previous work for POLA that was performed by others.

In addition to the scenario earthquake modeling, there were other differences between the current and previous examples. These consist of: (a) consideration of multiple discount rates, rather than a single rate, in the prior example; and (b) a modification of the site coordinates in the current example.

Because the objective of this analysis is solely to demonstrate the economic and risk evaluation procedure discussed above, the analysis contains certain simplifications that should be improved, to the extent possible, when applying the procedure to an actual port. These include:

- The analysis should consider more detailed characterization of faults in that could affect the hazards at the site, as well as local soil conditions and potential for ground failure due to liquefaction, slope instability, and surface fault rupture. The procedures recommended by the other investigators under this CSLC-USN project for marine oil terminals should be helpful for this purpose.

- The modeling of the seismic vulnerability of the wharf structure in this example is very simplified and should be improved. Again, the procedures in other chapters of this report should be helpful in this regard.

- It is preferable that the entire port be treated as a system. That is, instead of concentrating on only one component such as the wharf structure in this example, other components and their operational and physical interfaces should be addressed as well.

- The example addresses only one type of seismic risk reduction alternative – the selection of the level of seismic design acceleration to be considered for the wharf design. It does not consider that range of other seismic risk reduction alternatives that are available.

- Analyses of uncertainties and higher order economic losses are still very much in the research and development stage. Sensitivity analyses are desirable to overcome the belief that current models yield precise and accurate (rather than approximate) results.

- The example directly considers only ground shaking hazards. Other hazards that could be significant at a port or a marine oil terminal – such as liquefaction, slope instability, and surface fault rupture – should be considered in future applications of this procedure to an actual port or marine oil terminal.

Finally, it is important to emphasize that this example is for a hypothetical commercial container port for which the principal risks are economic losses due to excessive repair costs and loss of operations. Other possible risks from earthquakes, such as environmental risks, risks to life safety, etc. have not been considered. The extension of this risk analysis procedure to also address environmental risks at marine oil terminals is described in a later section.

Step 1: Define System and Components to be Evaluated The hypothetical facility in this demonstration application is the pile/wharf structure, embankment and dike shown in Figure 6-8. This wharf has a total length of 4,000 feet. It consists of a cast-in-place concrete flat-slab deck system supported on 24-inch diameter prestressed concrete piles that extend into the underlying rock embankment. The wharf is located in the Los Angeles – San Pedro area of southern California. Its site has a longitude of –118.28 degrees and a latitude of 33.74 degrees. This is close to but not identical to the site originally analyzed for POLA.

Immediately behind this structure is a zone of fills that is 75 ft. wide and is prone to isolated pockets of liquefaction. This zone is not critical to wharf operations, and prior investigation has shown that soil improvement costs to reduce liquefaction hazards to this area are greater than the economic risks associated with these hazards (i.e., repair costs and losses due to interruption of wharf operations). Therefore, a decision was made not to proceed with improvement of these soils. Accordingly, analysis of costs and risks due to liquefaction of these fills is not included in this demonstration application.

Step 2. Identify Permissible Seismic Risk Reduction Alternatives The seismic risk reduction alternatives considered in this example pertain to the selection of a design acceleration corresponding to the "Level 2 Earthquake" (L2E) motion for the seismic design of a major wharf structure. The L2E motion is defined as the level of earthquake ground shaking for which damage could occur, but impairment of port operations and other economic risks would be maintained at acceptable levels. It is noted that seismic performance requirements for this hypothetical wharf also require that the wharf be designed to resist a lower levels of shaking – termed the "Level 1 Earthquake" (L1E) motion – with no significant damage. In this example, the L1E motion was set equal to a constant multiple (0.533) of the L2E motion. The L2E and L1E ground motions are defined in terms of a peak horizontal ground acceleration (PGA) expressed as a fraction of gravity, g.

This demonstration example also assumes that the designation of the L2E motion for the design of this wharf is not mandated through regulation or code. Level 1 and Level 2 earthquake motions are minimum requirements specified by the criteria guidelines developed herein. This

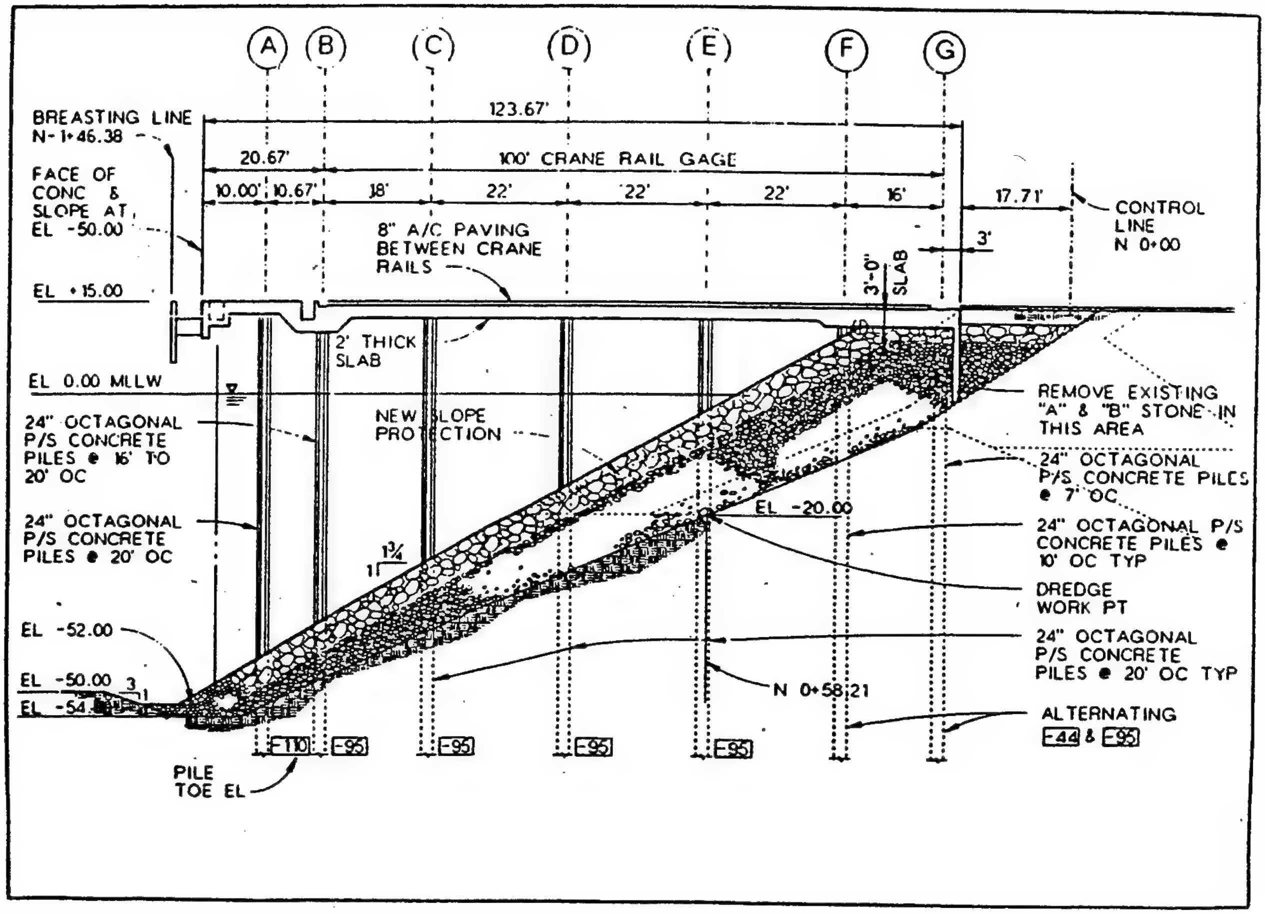


Figure 6-8
Cross Section of Wharf

example is a demonstration of the procedure to illustrate the effect of various seismic design levels. **As such it does not follow the criteria recommendations requiring use of a Level 1 and Level 2 earthquake motions and associated response strains.** In this demonstration example, all possible levels of L2E motion are evaluated in terms of their economic risk consequences to the wharf. Therefore, the example shows that definition of design level ground motions in terms of fixed probability levels may be overly conservative in some cases and unconservative in other cases. This will depend on the facility's location, seismic response characteristics, important seismic risks to be considered, and seismic performance requirements relative to these risks, as well as the seismologic and geologic characteristics of the surrounding region.

In this example, the seismic risk reduction alternatives consist of the seven design PGA levels for the L2E motions that are listed in Table 6-3. Based on these seven alternatives, interpolation was used to represent a continuum (from 0.0g to 0.60g) of seismic design alternatives. It is noted that the largest PGA induced at this site by any of the scenario earthquakes in this application is about 0.7g.

Table 6-3
Seismic Design Alternatives Considered For Demonstration Acceptable Risk Analysis Of Hypothetical Wharf

Seismic Design Alternative	PGA Level used to determine Lateral Design Force for Level 2 Earthquake (L2E) Motions
1	0.0 g (no seismic resistance built into wharf design).
2	0.24 g
3	0.30 g
4	0.37 g
5	0.45 g
6	0.50 g
7	0.60 g

Step 3: Define Multiple Scenario Earthquakes In this example, scenario earthquakes are established by *adapting* California data, models, and assumptions used by the United States Geological Survey (USGS) under their probabilistic National Hazard Mapping Project. USGS have worked jointly with their counterparts at the California Division of Mines and Geology

(CDMG) in order to develop data and models for representing California earthquakes. (see Frankel et al., 1996 and Cramer et al., 1996)

In this application, this process is used to develop over 20,000 scenario earthquakes throughout California that are consistent with the USGS and CDMG source models, maximum magnitude designations, activity rates, etc. These scenarios result from a random-walk analysis for a duration of 10,000 years. Ground motion attenuation equations are applied to each earthquake, in order to assess which earthquakes could cause damaging levels of ground shaking at the site being evaluated.

More specific information on the various types of earthquake sources that comprise this model, and the extent of the model that was considered can be provided. One of the significant sources for this example is an active fault that underlies a portion of the wharf.

Step 4: Estimate Site-Specific Seismic Hazards As previously noted, the only seismic hazard that is modeled in this demonstration application is ground shaking. Potential hazards from liquefaction, slope instability, and surface fault rupture are not considered

The USGS National Hazard Mapping program models ground motion attenuation by using the equations developed by Campbell et al. (1994), Boore et al, (1993, 1994a, and 1994b), and Sadigh et al.(1993). Results of these attenuation equations are equally weighted in accordance with a "logic tree" procedure. These investigators since updated their attenuation functions in the January/February, 1997 volume of *Seismological Research Letters*. For this demonstration analysis, ground motions are estimated by applying the Boore et al. (1997) attenuation functions for peak horizontal ground acceleration only, in which the wharf's site is represented by a NEHRP Type D site classification with an effective shear wave velocity of 250 m/sec. Uncertainties in these attenuation functions are not modeled, although procedures for so doing are available (see Werner et al., 1998). A more thorough evaluation could compare the diverse attenuation functions available and their uncertainties. Likewise, a more extended port study involving spatially dispersed components with diverse soil conditions would consider differences in soil amplification effects on the ground shaking at these diverse sites.

The Boore et al. (1997) relationship has the following form

$$\ln Y = b_1 + b_2(M_w - 6) + b_3(M_w - 6)^2 + b_5 \ln r + b_v \ln(V_S / V_A) \qquad \text{(6-1)}$$

where

$$r = \sqrt{r_{jb}^{\ 2} + h^2}$$

and

Y is the ground-motion parameter (spectral acceleration at a variety of natural periods or, for this, example, peak horizontal ground acceleration, in units of g)

b_1 is defined separately for strike-slip, reverse-slip, and mechanism-unspecified scenarios

M_w is the moment magnitude,

r_{jb} is the epicentral distance and h is the focal depth (both in km),

V_S is the average shear-wave velocity of the site soil materials in question (=250 m/sec), and b_3, b_5, b_V, and V_A are regression coefficients developed for a variety of periods of potential interest.

In this application, the following parameter values are used to compute peak ground acceleration in accordance with the above equation: b_1= -0.242 for all types of faults; b_2 =0.527; b_3= 0.0; b_5=0.778; h= 5.57 km; V_A= 1,396 m/sec; and b_v=-0.371.

The Boore et al. (1997) attenuation equation is used to compute PGAs at the wharf site for each scenario earthquake considered in the walk through analysis. A probabilistic seismic hazard analysis is then carried out according to the following procedure:

- PGA values in increments of 0.01 g are sorted in increasing order. For the ith PGA value, (denoted as $(PGA)_i$, the number of other PGAs with larger values is counted. This is represented as N'_i.

- The annual frequency of occurrence of PGA values **in excess of** $(PGA)_i$, denoted as υ'_i, is

$$\upsilon'_i = N'_i/10{,}000 \qquad (6\text{-}2a)$$

where 10,000 years is the total duration of the walk-through analysis for this example. Note that this frequency differs from the frequency of occurrence of PGA values **equal to** $(PGA)_i$, which is denoted as υ_i. If there are N_i samples of PGA values equal to $(PGA)_i$, then

$$\upsilon_i = N_i/10{,}000 \qquad (6\text{-}2b)$$

- The probability that $(PGA)_i$ is exceeded over an exposure time of T years is computed as

$$P(A \geq (PGA)_i)_T = 1 - \exp^{-\upsilon'_i T} \qquad (6\text{-}3)$$

Step 5: Implement Alternative Seismic Design/Strengthening Strategies for Individual Components within Overall System

1. **Implementation of Alternative Seismic Design Strategies** A preliminary seismic design of the wharf is carried out for each L2E design acceleration level listed in Table 6-3. Then, initial construction costs for each design were estimated. These are shown in Table 6-4, and are expressed as a multiple of an assumed baseline replacement cost of $65 million, which is the total replacement cost for the wharf when no seismic design is implemented (L2E acceleration = 0.0 g). Therefore, initial seismic outlays are the marginal costs of constructing a wharf designed to the range of non-zero CLE acceleration levels listed in Table 6-3.

2. **Seismic Vulnerability Assessment.** Assessment of the seismic vulnerability of this hypothetical wharf is based on linear and nonlinear pseudostatic analysis methods. This assessment uses only very preliminary information on potential embankment deformations, and does not include effects of soil-structure interaction. In addition, the possible beneficial effects of pinning action of the wharf's pile elements are neglected. A follow-up evaluation would be desirable to incorporate these potentially important effects.

The following discussion outlines considerations for estimating repair costs due to damage to each wharf design alternative that estimated by the seismic vulnerability analysis. The resulting repair cost model that is used for this demonstration analysis is also described. It is noted that this repair cost modeling for this demonstration analysis is based on a number of simplifying assumptions. When analyzing acceptable risks to an actual port, more detailed estimates of repair costs should be developed.

(a) Repair Considerations

The estimation of repair costs and times at an actual wharf should consider the anticipated damage modes, repair strategies, available labor, materials, and equipment for implementing the repairs, and repair strategies to minimize impacts on ongoing operations at undamaged sections of the wharf. These considerations for this particular hypothetical wharf are listed below.

TABLE 6-4
Initial Construction Costs For Various Seismic Design Alternatives

Seismic Design Alternative		Initial Seismic Construction Cost	
Number	L2E Design Acceleration	Total	As Multiple of Baseline Replacement Cost ($65 Million)
1	0.0 g	$ 0.0	0.00
2	0.24g	$ 0.7 M	0.011
3	0.30g	$ 2.2 M	0.034
4	0.37g	$ 3.3 M	0.051
5	0.45g	$ 4.9 M	0.075
6	0.51g	$ 5.2 M	0.08
7	0.60g	$ 10.4 M	0.16

- At PGAs above the design L2E acceleration level, it is estimated that the landward row of piles (i.e., the G row in Figure 6-8) will take the brunt of the seismic force, and will suffer the major damage. At these higher accelerations, damage is also anticipated at the F row of piles outboard from the dike. The pile damage is expected to be concentrated at the connection of the pile to the deck. However, at these high acceleration levels, it is estimated that ground deformation could cause additional damage in the form of spalling of the cover to the piles below grade. Although this additional damage is not expected to impair the structural integrity of the piles, the loss of concrete cover could lead to accelerated corrosion of the prestressing strands and the confinement steel.

- Possible repair strategies include: (a) excavation below the deck to expose the landward rows of piles; (b) repair of the connection between the piles and the deck, and also any spalling damage along the length of the pile, to prevent corrosion of the prestressing and reinforcing steel.; (c) backfill of the dike with rock to improve the dike's lateral stability; (d) installation of a cutoff wall; and (d) backfilling behind the wharf, preparation of a base for AC paving, and installation of the paving

- Repair work is estimated to occur over small lengths of the wharf (50-200 ft) in order to reduce operational constraints on the container wharf. Repair costs will not be very sensitive to this length.

- The duration of the repair effort is assumed to be roughly proportional to the number of crews assigned to the repair. In this example, it is assumed that two crews will work simultaneously to repair the wharf at different damage locations.

- It is assumed that the main variable in the repair model is the number of damaged piles, which accounts for about 20 percent of the total repair costs. Excavation and backfilling are assumed to require a minimum of one (1) work week (5 work days) per 100 foot section. At lower PGA levels, the time to repair piles does not generally exceed the time to perform the excavation and backfilling.

- If the underlying fault at the wharf generates an earthquake with surface rupture, it is assumed that repair costs will sharply increase. These repair costs are assumed to include costs for repair of the pile connections and for replacement of 800 feet of wharf (the spacing between expansion joints). Crane rails may need to be realigned, and to do so may require replacement of the wharf deck merely to provide adequate transition to allow the cranes to traverse across the misaligned section. It is estimated that one berth along the wharf will be out of service for about one year during reconstruction.

(b) Repair Times

Based on the above considerations, repair times are estimated from the following assumptions:

- At PGAs below the L1E design acceleration value, only a brief inspection period is required. No subsequent repair time is needed.

- At PGAs equal to the L1E design acceleration value, approximately 180 work days (8 calendar months) are required to complete the repairs.

- At PGAs slightly above the L2E design acceleration (L1E design acceleration + 0.02 g), approximately 200 work days (8 calendar months) are required to complete the repair. This estimate is assumed to be valid for all scenario earthquakes not involving significant surface fault rupture at the wharf.

- If significant fault rupture occurs at the wharf, approximately 260 work days (12 calendar months) are required to complete repairs. This would significantly impact other wharf operations.

(c) Overall Repair Cost Model

The resulting repair cost model for this hypothetical wharf is based on the above assumptions and considerations, together with regional construction rates adjusted to account for expected

access difficulties and restrictions. The repair cost model is summarized below, and is shown in Figure 6-9 for each seismic design alternative.

- At PGAs below the L1E design acceleration, there are no repair costs.

- At PGAs slightly above the L1E design accelerations, damage repair costs are five (5) percent of the baseline replacement cost of $65 million.

- At PGAs equal to the L2E design acceleration level, repair costs are six (6) percent of the baseline replacement cost.

- For each incremental 0.03 g increase in PGA above the L2E design acceleration level, repair costs increase by about 0.077 times the baseline replacement cost.

- Regardless of the dollar losses from ground shaking, serious surface fault rupture (leading to between one and three meters of permanent ground displacement) causes an additional repair/replacement cost of 18 percent of the baseline replacement cost, due to misalignment of the wharf face.

Step 6: Evaluate Seismic Performance of the Overall System Because only a single port component is considered in this example, analysis of secondary and higher order losses is limited to an analysis of possible direct losses of throughputs to the port. The complexity of even this business interruption loss analysis can be very significant (Morrison et al., 1986). For performing this analysis, one may consider such factors as: (a) the excess capacity of the port (the ability of other wharves to handle cargo); (b) the various types of cargo handled at the port (e.g., metal products, automobiles, cement, gypsum, and cement clinkers, ores scrap metal and other dry bulk, break-bulk, forest products, crude oil, refined petroleum products); (c) daily schedules, increased demand over time to the port facilities etc.; and (d) which of various stakeholders bears the secondary and higher order losses (e.g., shippers, the port itself, etc.) (Werner et al., 1998)

In this demonstration analysis, an upper bound estimate of business interruption losses is developed. This estimate is based on the following assumptions: (a) the wharf handles 3,300 TEU of container cargo during each work day; (b) the port will lose $26 for each TEU not handled due to earthquake damage; and (c) the duration of the business interruption loss will be directly proportional to the primary losses (repair costs) incurred due to earthquake damage to the wharf; and (d) this constant of proportionality considers that if the required repair costs following a given earthquake (L_i) equal $0, the duration of the business interruption (D_{BI}) will be zero days, and if the required repair cost equals the total baseline replacement cost ($R_c = \$65,000,000$), the duration of the business interruption will equal 280 days, i.e.,

$$D_{BI}\,(days) = \frac{280 L_i}{R_c}$$

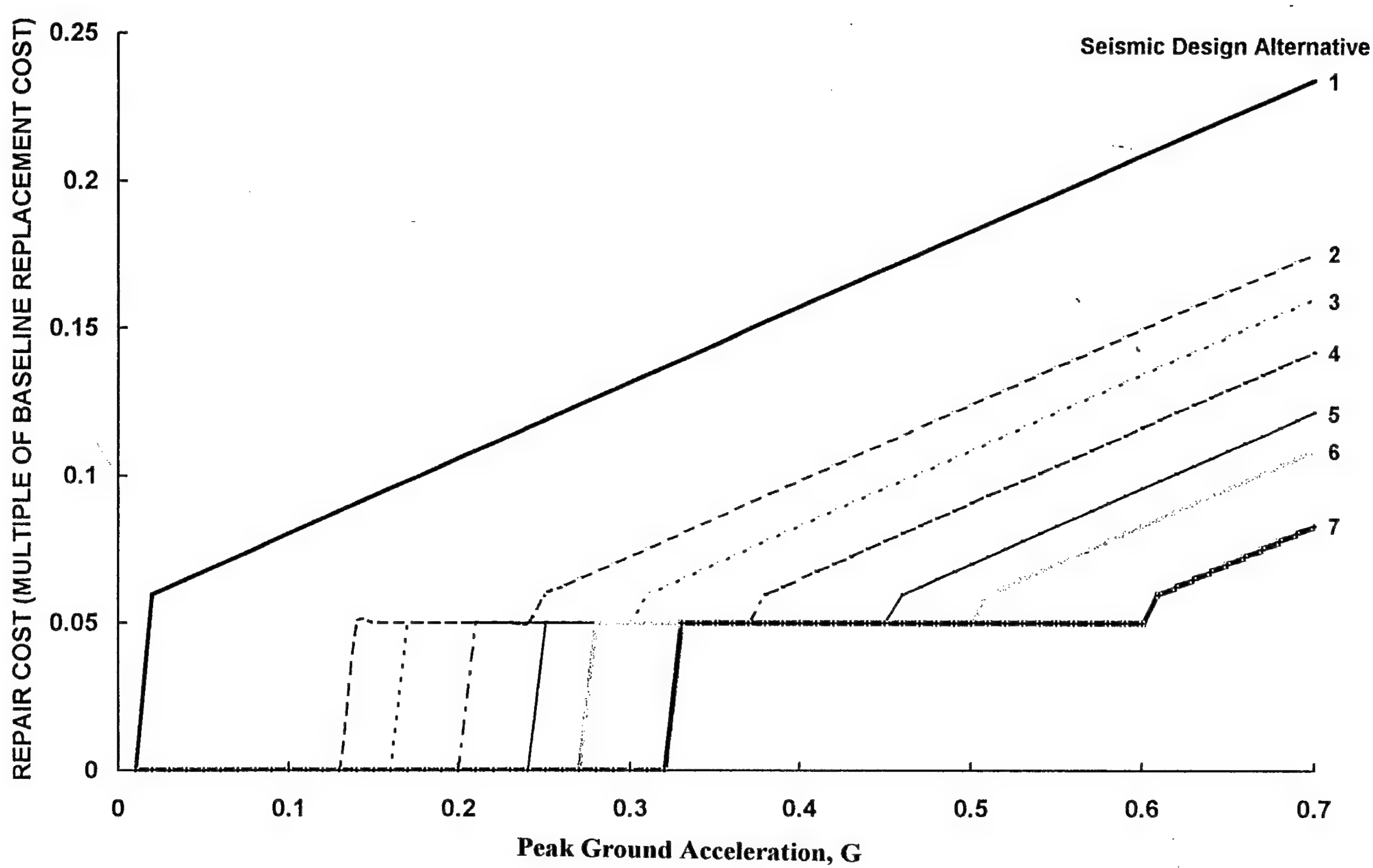


Figure 6-9
Repair Cost Model for Wharf Demonstration Analysis

Therefore, the total number of TEUs not handled at the port due to earthquake damage is 3,300 x $280L_i$, and the losses due to earthquake-induced business interruption, L_{BI}, is

$$L_{BI} = \$26x(\$3{,}330x\frac{280L_i}{R_c}) = \$24{,}024{,}000x\frac{L_i}{R_c}$$

Substituting $R_c = \$65{,}000{,}000$ into the above expression, the cost of business interruption becomes

$$L_{BI} = \$24{,}024{,}000 \; x \; L_i \,/\, \$65{,}000{,}000 = 0.37 \; x \; L_i \qquad (6\text{-}4)$$

The average annualized value of the business interruption cost, $L_{BI,TOT}$ is estimated by substituting the average annualized value of repair costs, $L_{R,TOT}$, for L_i in the above equation, i.e.,

$$L_{BI,TOT} = 0.37 \; x \; L_{R,TOT} \qquad (6\text{-}5)$$

where the computation of $L_{R,TOT}$ is described in Step 7.

It is noted that the above estimate of business interruption losses is an upper bound because the inherent assumptions ignore: (a) the likelihood that the some if not all of the entire wharf will be operable after almost all earthquakes, and so will permit average to peak loads virtually whenever they are available; and (b) double-counting considerations, e.g., the transportation system for the wharf may also be damaged and wharf damage will therefore not necessarily be responsible or solely responsible for business interruption losses incurred.

Step 7. Assess Seismic Risks and Modify Component Designs if Appropriate The seventh step contains three major substeps: (a) development of risk and decision calculations for the design alternatives, in terms of key performance criteria; (b) selection of the decision alternative(s) that best meet these criteria; and (c) review these selections(s) and their rationale with the public. The following calculations of least costs and variances illustrate the application of Substep a.

Step 7 Least Cost Calculations For each of the seven seismic design alternatives considered in this example, calculation of overall mean life-cycle costs involves the following three steps: (a) calculation of the average annualized loss; (b) calculation of the present value of the losses; and (c) adding this present value of the losses to the initial construction costs to derive the overall mean life-cycle costs. These steps are further described below.

(a) Substep 7-1: Calculation of Average Annualized Value of Repair Cost for Each Seismic Design Alternative

To carry out this step, it is necessary to first calculate:

- The annual frequency of occurrence of each PGA level. As noted in Step 4, this quantity is computed from Equation 6-2b, and is denoted as υ_i for the ith PGA level $(PGA)_i$

- The repair costs associated with each PGA level for each design alternative. The estimation of this cost should consider possible types of repairs needed for each PGA level, in accordance with the results of the seismic vulnerability analysis. The repair cost at the i^{th} PGA level is denoted as ℓ_i.

From this, the average annualized repair cost for the decision alternative, $L_{R,TOT}$ is computed as

$$L_{R,TOT} = \sum_{i}^{NA} \upsilon_i \ell_i \tag{6-6}$$

where NA is the total number of incremental PGA values considered in this example.

This formulation uses "frequencies" rather than probabilities, because probabilities can underestimate average annualized losses[1]. To illustrate, assume that the number of accidents by drivers in a neighborhood averages 3 per year, with an average cost of $1,200 per accident. Hence, the average annualized cost is 3 x $1,200, = $3,600 per year. Using probabilities, one may find that in 90 percent of the years at least one traffic accident occurs. Ignoring the probabilities of occurrence of 2, 3, 4, or more accidents in a year, one might erroneously conclude that the average annualized loss is 0.9 x $1200, or $1080. In general, the use of frequencies of occurrence is preferable to probabilities in regions of higher seismicity with more frequent earthquakes and/or strong ground motion levels.

To illustrate how the average annualized value of repair costs is calculated, one might examine design alternative 5 (L2E acceleration = 0.45g). Table 6-5 summarizes these calculations. It begins with a PGA of 0.25g since this is slightly above the L1E acceleration for Design Alternative 5. Below this level, it is assumed that no significant damage occurs. It should further be noted that the frequency of occurrence is not—as might be expected—monotonically decreasing as PGAs increase. This is chiefly a result of the Monte Carlo sampling method employed. Since almost 14,000 earthquake scenarios generate PGAs of 0.01 g or greater for the 10,000 year time frame simulated, the number of simulations is statistically robust. Only for small probabilities should the simulation program consider longer time frames and many more uncertainties; instead, most of the emphasis of the uncertainty evaluation should be on the modeling itself.

(b) Substep 7-2: Calculation of Present Value of Losses

As noted above, the discount rate multiplier for a constant dollar value discount rate j (based on annul loss) and a time of exposure of T years, denoted as $R_{j,T}$, is computed from the following equation:

[1] This underestimation will occur unless probabilities of two occurrences, three occurrences, and so on are considered.

$$R_{j,T} = \frac{1-(1+j)^{-T}}{j} \quad (6\text{-}7)$$

Using the above equation for a 50-year time of exposure, the discount rate multipliers associated with a range of discount rates are computed, as shown in Table 6-6. This table shows that, as discount rates increase, the impacts of reducing earthquake losses decreases.

Once a discount rate is established and the corresponding discount rate multiplier is computed, the present value of the total loss, including repair costs plus business interruption losses, is computed as:

$$L_{PV} = R_{j,T}(L_{R,TOT} + L_{BI,TOT}) \quad (6\text{-}8)$$

Table 6-7 illustrates the computation of the total mean life-cycle cost for Design Alternative 5, based on discount rates of 1% and 7%, respectively. This proceeds as follows:

- The last line of Table 6-5 has shown that the average annualized repair cost value for Design Alternative 5 (computed using Equation 6-6), as a ratio of the baseline replacement cost for the wharf, is

$$L_{R,TOT} = 0.00075 \quad (6\text{-}9)$$

- Step 6 has shown that, for this hypothetical wharf, the average annualized business interruption loss for Design Alternative 5, L_{BI} , is 37 percent of the average annualized repair cost value, $L_{R,TOT}$. Therefore, the business interruption loss is

$$L_{BI,TOT} = 0.37L_{R,TOT} = 0.37 * 0.00075 = 0.00028 \quad (6\text{-}10)$$

From this, the total loss, L'_{TOT}, (including both repair costs and business interruption losses) is

$$L'_{TOT} = L_{R,TOT} + L_{BI,TOT} = 0.00075 + 0.00028 = 0.00103 \quad (6\text{-}11)$$

Table 6-5
Average Annualized Value of Repair Cost For Design Alternative 5 (Cle = 0.45g)

PGA, g	Frequency of Occurrence, υ_i	Repair Cost at i[th] PGA Level, ℓ_i	Annualized Repair Cost at i[th] PGA level = $\upsilon_i \ell_i$
0.25	0.0002	0.05	0.00001
0.26	0.0013	0.05	0.00007
0.27	0.0004	0.05	0.00002
0.28	0.0006	0.05	0.00003
0.29	0.0015	0.05	0.00008
0.30	0.0022	0.05	0.00011
0.31	0.0001	0.05	0.00001
0.32	0.0004	0.05	0.00002
0.33	0.0002	0.05	0.00001
0.34	0.0002	0.05	0.00001
0.35	0.0002	0.05	0.00001
0.36	0.0010	0.05	0.00005
0.37	0.0002	0.05	0.00001
0.38	0.0000	0.05	0.00000
0.39	0.0002	0.05	0.00001
0.40	0.0001	0.05	0.00001
0.41	0.0003	0.05	0.00002
0.42	0.0014	0.05	0.00007
0.43	0.0004	0.05	0.00002
0.44	0.0012	0.05	0.00006
0.45	0.0002	0.05	0.00001
0.46	0.0000	0.06	0.00000
0.47	0.0000	0.0626	0.00000
0.48	0.0004	0.0651	0.00003
0.49	0.0000	0.0677	0.00000
0.50	0.0000	0.0702	0.00000
0.51	0.0001	0.0728	0.00001
0.52	0.0000	0.0754	0.00000
0.53	0.0000	0.0779	0.00000
0.54	0.0000	0.0805	0.00000
0.55	0.0002	0.0830	0.00002
0.56	0.0000	0.0856	0.00000
0.57	0.0001	0.0882	0.00001
0.58	0.0000	0.0907	0.00000
0.59	0.0000	0.0933	0.00000
0.60	0.0001	0.0958	0.00001
0.61	0.0000	0.0984	0.00000
0.62	0.0000	0.1010	0.00000
0.63	0.0002	0.1035	0.00002
0.64	0.0001	0.1061	0.00001
0.65	0.0002	0.1086	0.00002
0.66	0.0000	0.1112	0.00000
0.67	0.0000	0.1138	0.00000
0.68	0.0000	0.1163	0.00000
0.69	0.0001	0.1189	0.00001
0.70	0.0000	0.1214	0.00000
Sum (= $L_{R,TOT}$)			0.00075

TABLE 6-6
Discount Rate Multipliers For 50 Year Exposure Time

Discount Rate, j	Discount Rate Multiplier, $R_{j,50}$ (for annual loss)
1%	39.2
2%	31.4
3%	25.7
4%	21.5
5%	18.3
6%	15.8
7%	13.8
8%	12.2
9%	11.0
10%	9.9

- Table 6-6 shows that the present value of this total loss is obtained by multiplying it by 39.2 for a real discount rate of 1% and by 13.8 for a real discount rate of 7%. Therefore, the present value of the losses for Design Alternative 5 is

$$L_{PV} = 39.2 * 0.00103 = 0.0404 \quad \text{for a discount rate of } 1\%$$

and

$$L_{PV} = 13.8 * 0.00103 = 0.0142 \quad \text{for a discount rate of } 7\%$$

The values of L_{PV} for all design alternatives are shown in Table 6-7.

(c)Substep7- 3: Determination of Overall Mean Life-Cycle Costs

The mean value of the total life cycle cost, C_{LC}, is the sum of the present value of losses, L_{PV}, plus the initial construction cost, C_C, i.e.,

$$C_{LC} = L_{PV} + C_C \tag{6-12}$$

For Design Alternative 5, the initial construction cost is 0.075 times the baseline replacement cost of the wharf. Therefore, C_{LC} is computed as

$$C_{LC} = 0.075 + 0.040 = 0.115 \text{ for a discount rate of } 1\% \qquad (6\text{-}13)$$

and

$$C_{LC} = 0.075 + 0.014 = 0.089 \text{ for a discount rate of } 7\% \qquad (6\text{-}14)$$

The total life cycle costs for Design Alternative 5 and also for the other design alternatives are shown in Table 6-7.

Table 6-7
Illustrative Calculations of Mean Life Cycle Costs for the Seven Design Alternatives

Cost*	Alt.. 1 (L2E= 0.0g)	Alt. 2 (L2E = 0.24g)	Alt. 3 (L2E = 0.30g)	Alt. 4 (L2E = 0.37g)	Alt. 5 (L2E = 0.45g)	Alt. 6 (L2E = 0.50g)	Alt. 7 (L2E = 0.60g)
Average Annualized Repair Cost	0.0482	0.00205	0.00150	0.00103	0.00075	0.00063	0.00037
Business Interruption Loss	0.0178	0.00076	0.00056	0.00038	0.00028	0.00023	0.00014
Total Average Annual Loss	0.0660	0.0028	0.0021	0.0014	0.0010	0.0009	0.0005
Present Value of Losses (Discount Rate =1%)	2.588	0.110	0.080	0.055	0.040	0.034	0.020
Present Value of Losses (Discount Rate =7%)	0.911	0.039	0.028	0.019	0.014	0.012	0.007
Initial Seismic Construction Cost	0.00	0.011	0.034	0.051	0.075	0.08	0.16
Total Mean Life-Cycle Cost (Discount Rate = 1%)	2.588	0.121	0.114	0.106	0.115	0.114	0.180
Total Mean Life-Cycle Cost (Discount Rate = 7%)	0.911	0.050	0.062	0.070	0.089	0.092	0.167

*Costs given as multiple of baseline replacement cost for wharf configuration with no seismic design (= $65,000,000).

Figures 6-10 and 6-11 illustrate the development of total mean life-cycle costs for each design alternative, as the sum of the initial seismic construction cost (i.e., the construction cost over and above the construction cost if no seismic design is implemented) plus the present value of the losses. These results are provided in Figures 6-10 and 6-11 for discount rates of 1 % and 7 % respectively.

Figure 6-12 compares the mean life-cycle costs for discount rates of 1 % and 7 %. This comparison demonstrates the sensitivity of the results to the discount rate selected. For example, Figure 6-12 shows that, if a discount rate of 1 % is selected, Design Alternative 4 has the most favorable mean life-cycle cost whereas, if a discount rate of 7 % is selected, Design Alternative 2 has the most favorable cost. The figure also shows that, for a given design alternative, the mean life-cycle cost decreases as the selected discount rate increases – i.e., higher discount rates will generally reduce the importance of seismic risk reduction activities.

Figures 6-10 to 6-12 also show that Design Alternative 1 (no seismic design) has very large mean life-cycle costs as compared to the other design alternatives. This is due to the large present-value losses estimated for that alternative. Of the remaining alternatives, Design Alternative 6 (L2E design acceleration = 0.6 g) has the next highest mean life-cycle cost, due to its large initial seismic construction cost. The differences in mean life-cycle cost among Design Alternatives 2 through 5 are relatively minor if a discount rate of 1 % is selected, and are more pronounced when a discount rate of 7 % is considered.

Step 7 Variance Calculations Supplementing this least-cost analysis is an analysis of the variance of losses. As stated already, investments do not aim merely at the highest rate of return. To do so would be to ignore the volatility of investments, as represented by the variance or standard deviation of the losses. Therefore, whereas minimizing the least cost represents a maximization of the return of the investment in seismic risk reduction of this hypothetical wharf, reducing the variance and standard deviation of the losses is also prudent, from the standpoint of reducing the riskiness or volatility of the investment. A careful investor would consider both of these aspects when evaluating a potential investment. For example, junk bonds often have high rates of return; however, because of their extreme volatility, they are often not considered to be a good investment. Insurance purchase, hedging, portfolio diversification, and other activities are used in investing in order to reduce the volatility of investments. (Bernstein, 1996).

In this analysis, the variance of initial construction costs has not been estimated. Instead, the analysis confines itself to the calculation of variance and standard deviation of the losses. Also, it is not necessary to calculate the present value of the variance or standard deviation in order to demonstrate the relative volatility and riskiness of the various design alternatives. This is because the present value of variance and standard deviation is simply a linear multiple of the variance and standard deviation.

The variance of the earthquake losses for this example, σ^2, is computed as:

$$\sigma^2 = \sum_{i=1}^{NA} \frac{\upsilon_i}{\upsilon_{TOT}} \left(L'_i - L'_{TOT}\right)^2 \tag{6-15}$$

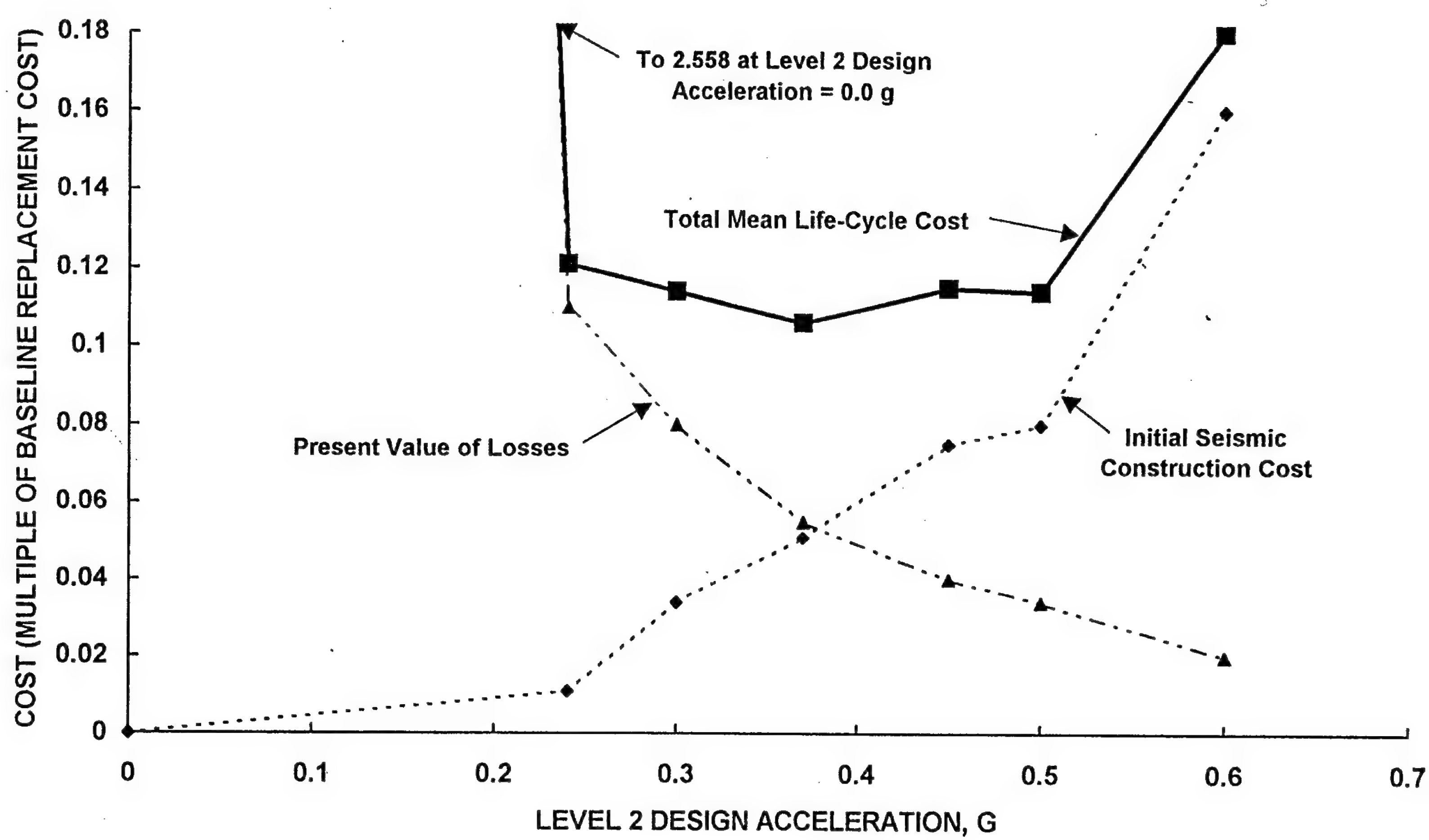


Figure 6-10
Development of Mean Life-Cycle Costs for Wharf Demonstration Analysis: Discount Rate = 1%

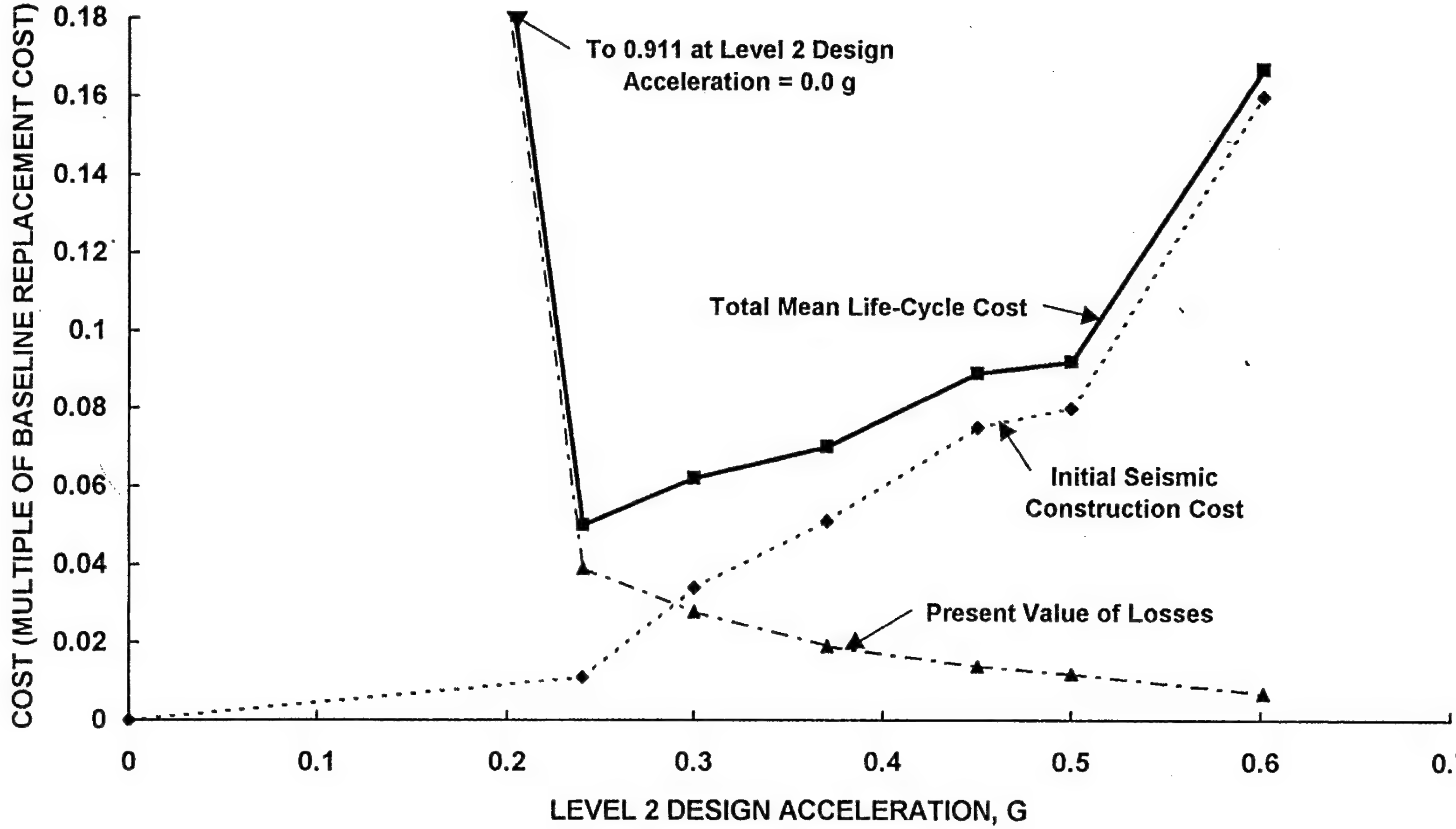


Figure 6-11
Development of Mean Life-Cycle Costs for Wharf Demonstration Analysis: Discount Rate = 7%

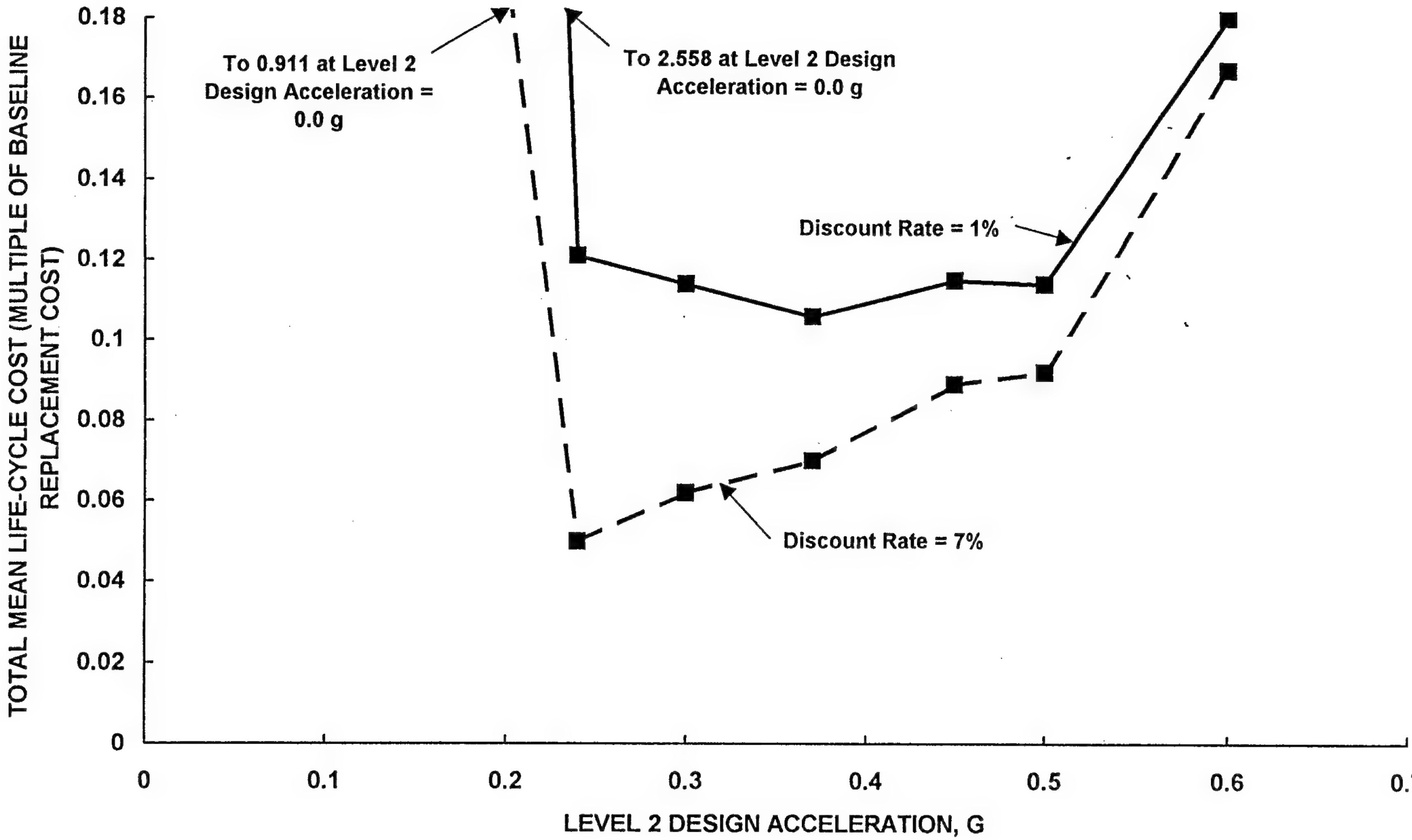


Figure 6-12
Effect of Discount Rate on Mean Life-Cycle Costs for Wharf Demonstration Analysis

where

NA = total number of PGA increments considered in the analysis

υ_i = annual frequency of occurrence of ith PGA level (Equation 6-2b)

$\upsilon_{TOT} = \sum_{i-1}^{NA} \upsilon_i$ = total annual frequency of occurrence of all PGA levels

L'_i = total loss due to ith PGA level including repair costs and business interruption losses

L'_{TOT} = average annualized value of total loss including repair costs and business interruption

The standard deviation of the earthquake losses is

$$\sigma = \sqrt{\sigma^2} \tag{6-16}$$

To illustrate the use of Equation 6-15 and 6-16, consider Design Alternative 5, and a PGA level of 0.01 g, whose parameters are as follows:

L'_{TOT} = average annualized loss including repair costs plus business interruption loss = 0.00103, expressed as a multiple of the baseline wharf replacement cost), as computed using Equation 6-8

υ_i = frequency of occurrence of PGAs with value of 0.01 g = 0.6809

υ_{TOT} = total frequency of occurrence of all PGA values = 1.3929

L'_i = total loss at PGA of 0.01 g = 0.0

Therefore the variance increment for this PGA level is

$$\sigma_i^2 = \frac{\upsilon_i}{\upsilon_{TOT}}(L'_i - L'_{TOT})^2 = \frac{0.6809}{1.3929}(0 - 0.00103)^2 = 5.19x10^{-7}$$

Similar calculations can be carried out for each of the other PGA levels. Then, the variance increments for all of the PGA levels are summed to obtain the total variance (which turns out to be 5.76×10^{-5} for Design Alternative 5. The resulting value of the standard deviation of the losses for this alternative is

$$\sigma_i = \sqrt{5.76x10^{-5}} = 7.59x10^{-3}$$

Table 6-8 and Figure 6-13 summarizes these estimates for standard deviation for all seven alternatives. This table and figure show that the standard deviation decreases as the CLE design acceleration increases. Thus, increasing the CLE design acceleration for this hypothetical wharf reduces the riskiness of the seismic performance of the wharf and the resulting volatility in the investment in the wharf's seismic risk reduction. Table 6-8 and Figure 6-13 also show that Design Alternative 1 (CLE design acceleration = 0.0 g) clearly has the largest standard deviation, demonstrating the extreme riskiness of the no seismic design option for this wharf facility.

Table 6-8
Standard Deviations for the Seven Seismic Design Alternatives

Seismic Design Alternative	Level 2 Design Acceleration	Standard Deviation, σ, x10^{-2} (Multiple of Baseline Replacement Cost)
1	0.0 g	5.190
2	0.24 g	1.455
3	0.30 g	1.192
4	0.37 g	0.950
5	0.45 g	0.759
6	0.50 g	0.685
7	0.60 g	0.507

Conclusion from Demonstration Application

The purpose of this demonstration analysis has been to illustrate the application of the acceptable-risk procedure to a commercial container wharf for which the primary risks of concern are earthquake-induced economic losses. The analysis was based on a random-walk evaluation that involved over 20,000 scenario earthquakes occurring over a 10,000 year time frame.

By necessity, the analysis entailed certain limitations in the treatment of the seismic hazards, in the modeling of the seismic vulnerability of the wharf, and in the estimation of repair costs

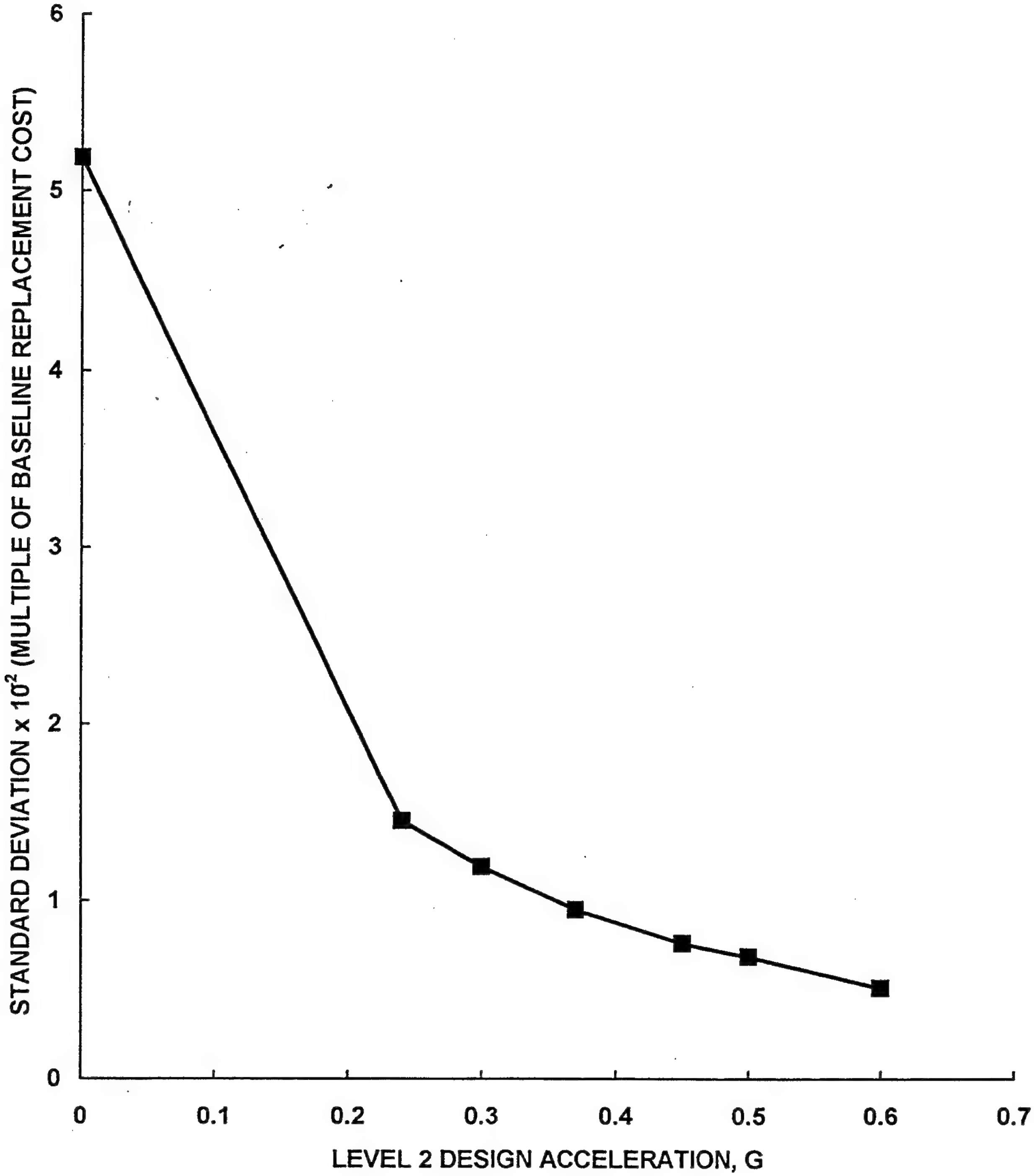


Figure 6-13
Standard Deviation of Losses for Wharf Demonstration Analysis

and business interruption losses. These simplifications may not be fully appropriate when this procedure is applied to an actual port, for use in guiding the subsequent selection of a seismic risk reduction strategy. It is noted that the acceptable risk evaluation approach can accept models with whatever level of sophistication is deemed appropriate by the user. Whatever degree of model sophistication is employed, the user should consider uncertainties in the models and the input data when interpreting the acceptable-risk analysis results for decision-making purposes.

Even though simplified models have been used, this demonstration analysis has clearly illustrated the applicability of the acceptable-risk method as a seismic risk reduction decision-making tool. The analysis results have also shown the following clear trends:

- The risk analysis results are sensitive to the discount factor that is selected.

- The mean-variance approach that is incorporated into the acceptable risk procedure enables the user to assess alternative seismic risk reduction options from the standpoint of an investor concerned not only with optimizing the yield of his investment in seismic risk reduction (i.e., examining the relative mean life-cycle costs of the various risk reduction alternatives), but also with maintaining tolerable levels of riskiness or volatility of his or her decision (by examining how the standard deviations of the earthquake losses differ among the various alternatives).

- For this example, the no seismic-design option was clearly shown to be extremely unfavorable, based on its very high values of mean life-cycle cost and standard deviation of earthquake losses.

- This example was intended to illustrate the application of the acceptable risk procedure and not to give specific guidance on cost-effective seismic design acceleration levels.

Application To Marine Oil Terminals

The demonstration application of the acceptable risk procedure that is described in the previous section has shown how the procedure can be used to assess economic risks due to earthquake damage at a commercial container port. This section describes how this same procedure can be used by a regulatory agency (i.e., CSLC) to assess various seismic risk reduction alternatives new or existing marine oil terminals. However, the performance criteria to be considered by the regulatory agency for marine oil terminals will differ from those of port decision-makers for a commercial container port. For a marine oil terminal, the primary risks of concern to the agency will be the environmental risks due to release of oil products into the surrounding waterway during an earthquake. However, cost would still be a factor from the standpoint of the practicality of implementing the regulations once they are in place. Therefore, a suitable balancing of these costs and risks is needed.

To describe the applicability of this procedure to marine oil terminals, this section is organized into two parts. The first part summarizes how the previously described steps of the

procedure can accommodate consideration of both economic and environmental risks. The second part outlines a qualitative application of the procedure to evaluate these risks for a marine oil terminal.

Extended Procedure

In this extended procedure, Steps 1 through 5 are identical to those described and illustrated above in the section titled "Expanded Economic Analysis To Include Risk". Steps 6 and 7 are modified as described below.

Step 6: Evaluate Seismic Performance of Overall System Step 6 evaluates the seismic performance of each alternative system configuration established in Step 5, when each configuration is analyzed for each earthquake that occurs during each year of the walk-through established in Step 4. The results of each seismic performance evaluation for each system configuration and each earthquake should indicate: (a) whether the marine oil terminal system has been damaged; and, if so: (b) the present value of total losses due to this damage (sum of initial construction costs from Step 5 plus repair costs, business interruption losses, oil spill costs and any higher order economic losses that can be assessed); and (c) whether this damage has led to a release of hazardous materials, quantification of the size of the release, and whether it exceeds CSLC acceptable spill volumes (in excess of 1,200 barrels).

Note the cost of an oil spill was shown above to be high and to involve not only direct cleanup costs but also costs of damage to the shoreline and environment and third-pary costs. These costs must be included

Step 7: Assess Seismic Risks and Modify Component Designs if Appropriate Step 7 carries out a reliability assessment of each alternative system configurations, based on the walkthrough analysis of scenario earthquakes that has a duration of 10,000 years. The end results of the analysis should provide the following information: (a) the present value of the total economic losses incurred by the system alternative over the 10,000 year duration; and (b) the "reliability" of each alternative – which is an assessment of the design alternative's potential for limiting the release of oil during an earthquake to an acceptable volume mandated by CSLC; and (c) the "risk" associated with each design alternative – which is an assessment of the potential that the design alternative will experience earthquake-induced oil spillage that will exceed CSLC acceptable volumes (i.e., the risk is the converse of the reliability). The focus here is on the risk and size of an earthquake-induced oil spill. Decision-making pertaining to the selection of an appropriate system alternative is based on prudent management of this risk. This reliability and risk assessment process is illustrated below.

Illustrative Application

To illustrate this process for a given marine oil terminal system, suppose that ten different seismic design alternatives have been developed for the various oil terminal components, and that the seismic performance of each alternative has been evaluated for a suitable number of scenario earthquakes. Also, suppose that this evaluation was in the form of a walk-through analysis with a duration of 10,000 years, and that acceptable seismic performance of the terminal system is defined in terms of limiting the volume of oil released during an earthquake to 1,200 barrels.

Finally, let us consider that the application of Steps 6 and 7 to each design alternative provides the following results: (a) the present value of the total mean life-cycle costs due to earthquake damage to the system over the 10,000 year duration; and (b) the "risk" associated with each design alternative, which is number of times during the walk-through when the system failed due to earthquake damage (i.e., more than 1,200 barrels of oil were released), divided by the 10,000 year duration of the walk-through. In addition, the "reliability" of each alternative is computed, which is the number of times during the walk-through when the system did not fail due to earthquake damage (i.e., less than 1,200 barrels of oil were released) divided by the 10,000 year duration of the walk-through. (Note that reliability = 1.0 – risk). Let us also assume that these results are as follows:

Alternative	Cost[2]	Reliability	Risk
1	\$4.3M	9,996/10,000	4/10,000
2	\$3.7M	9,990/10,000	10/10,000
3	\$5.5M	9,995/10,000	5/10,000
4	\$6.7M	9,997/10,000	3/10,000
5	\$4.5M	9,995/10,000	5/10,000
6	\$3.5M	9,991/10,000	9/10,000
7	\$3.9M	9,991/10,000	9/10,000
8	\$4.8M	9,993/10,000	7/10,000
9	\$5.3M	9,994/10,000	6/10,000
10	\$5.6M	9,996/10,000	4/10,000

A plot of the costs vs. risk for each alternative (Figure 6-14) shows that System Alternatives 1, 4, and 6 represent the most favorable cost–risk combinations. Alternative 4 is the lowest risk and highest cost option, Alternative 6 is the highest risk and lowest cost option, and Alternative 1 is a middle ground between these two extremes.

These cost vs. risk results provide information that can be used to guide the establishment of an appropriate design alternative for the marine oil terminal. This will depend on the acceptability of alternative levels of cost and risk that may be experienced. Input from various stakeholders and interveners may bean important element of this decision process.

[2] This cost is the total mean life-cycle cost, which is calculated as illustrated above.

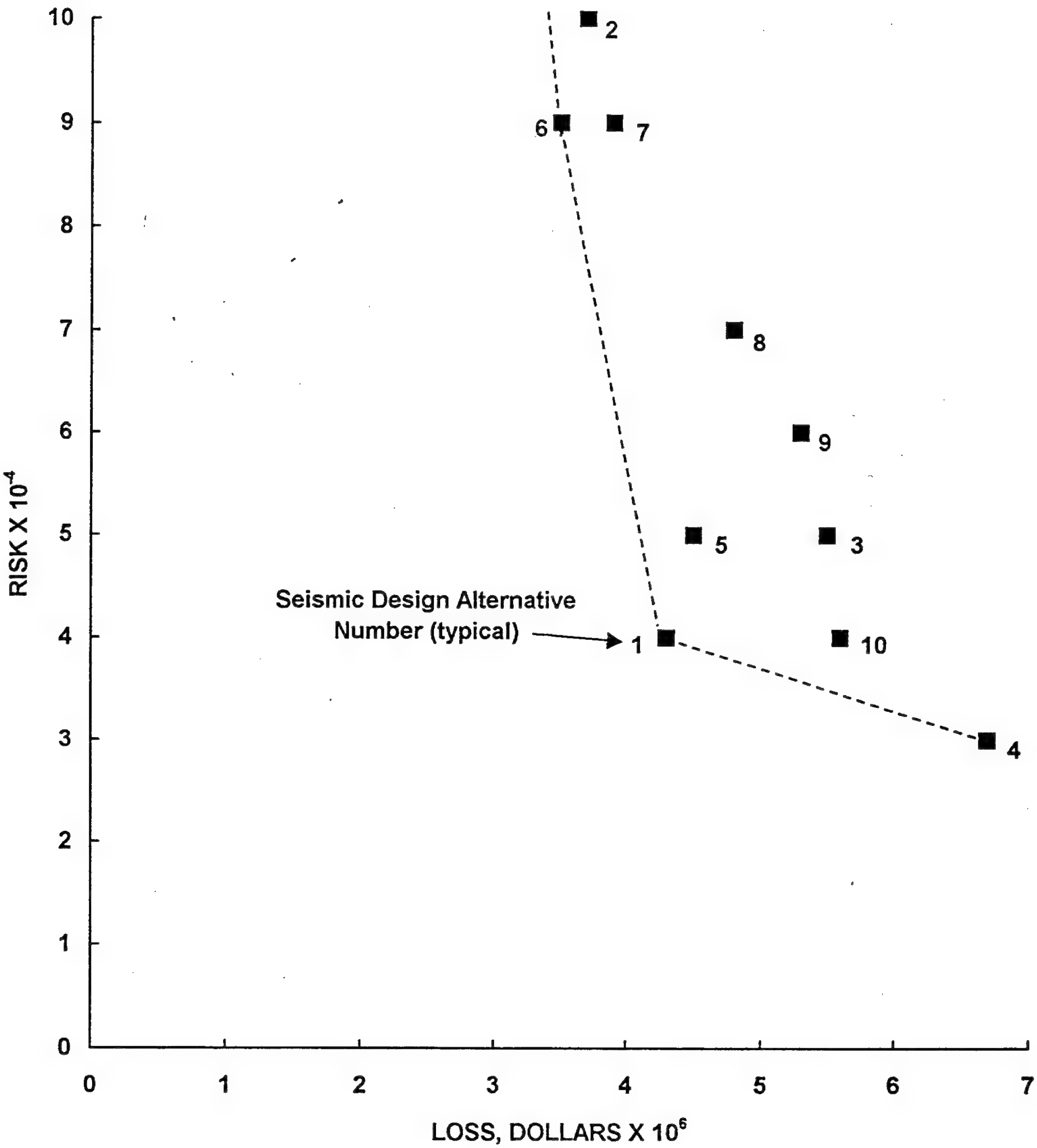


Figure 6-14
Form of Results: Acceptable Risk Analysis of Marine Oil Terminal

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CHAPTER 7 RELIABILITY

Introduction

This chapter is based on some of the work reported in detail in a technical report for the Navy Facilities Engineering Service Center by Ferritto and Putcha (1995b). This chapter presents an approach for evaluating the seismic reliability of a typical element of waterfront construction, wharves, but it is applicable to all types of construction. The reliability evaluation of a structure for various limit states, especially when these limit states are non-linear, is a complex problem by itself. This becomes even more involved when the structure is subjected to seismic excitation. A good amount of work has been done in the general area of seismic reliability analysis and the reader may refer, among others, to work done by Hwang et al.(1987), Hwang and Jaw(1990), Ang (1990), Tung and Kermidjian (1991), Moller and Rubinstein (1992), Hwang and Hsu (1993), and Wen et al. (1994) . These studies dealt with structures such as buildings, water tanks and nuclear power plants. Some of these studies in the literature , for example, the study by O'Connor and Ellingwood (1987) also dealt with reliability of non-linear structures under seismic loading. In the work by O'Connor and Ellingwood (1987) reference was also made to an earlier equivalent static analysis used for the reliability analysis of structures subjected to seismic forces by Ellingwood et al. (1980). The corresponding safety indices, β, were given along with the probability of failure, P_f, for each of the limit state. An important point to be noted is that the number of specific studies on the seismic reliability analysis of waterfront construction involving soil-structure interaction problems like wharves reported in the literature is limited.

Reliability Analysis - General Methodology

The reliability analysis methodology that is being proposed is general in nature for all structures subjected to seismic forces even though it is discussed with reference to wharves. The probability of failure of a wharf can be evaluated for each of the applicable limit states such as strain or ductility limit exceedance and yielding of piles, excess lateral displacement, etc. Then the bounds on the probability of failure of the wharf can be established, if need be, using the methods proposed by Ang and Tang (1984). The limit state function of a wharf is given by

$$g = R - L \tag{7-1}$$

where,

R = component of resistance capacity
L = component applied load

The above limit state uses the basic premise that the probability of failure is defined as:

$$P_f = P(g < 0) \tag{7-2}$$

$$P_f = P(R < L) \qquad (7\text{-}3)$$

Once P_f is calculated then the reliability can be evaluated from the following equation as:

$$\text{Reliability} = 1 - P_f \qquad (7\text{-}4)$$

It has been common practice presently to express reliability in terms of a reliability index β, which is expressed as,

$$\beta = \Phi^{-1}\left(1 - P_f\right) \qquad (7\text{-}5)$$

Where Φ^{-1} is the inverse of a standard normal cumulative distribution function. To be specific, β is the First Order Second Moment Reliability index, defined as the minimum distance from the origin of the standard, independent normal variable space to the failure surface as discussed in detail by Hasofer and Lind (1974), Ellingwood et al. (1980), and Ang and Tang (1984). The above relation is exact if the limit state function is linear and all probability distributions are jointly normal or lognormal.

There have been several applications of the First Order Second Moment (FOSM) and also the Advanced First Order Second Moment (AFOSM) methods in the literature, Ayyuba et al. (1984), Ellingwood et al. (1980), Galambos et al. (1978a), Galambos (1978b), and Hoeg et al. (1974), to name a few.

The safety index β is also expressed as,

$$\beta = \frac{\bar{g}}{\sigma_g} \qquad (7\text{-}6)$$

where,

$$\bar{g} = g\left(\bar{X}_1, \bar{X}_2, -----\bar{X}n\right) \qquad (7\text{-}7)$$

$$\sigma_g^2 = \Sigma\ (\partial g/\partial X_i)^2\ \sigma^2_X \qquad (7\text{-}8)$$

where the bar over the variable indicates the mean value. The partial derivatives are evaluated at the corresponding mean value of the variable. If g is defined by equation 7-1 then, for R and L being normal variables, β can be expressed as,

$$\beta = \frac{\overline{R} - \overline{L}}{\sqrt{\sigma_R^2 + \sigma_L^2}} \tag{7-9}$$

If R and L are assumed to have lognormal distribution then β can be expressed as,

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{L}}\right)}{\sqrt{V_R^2 + V_L^2}} \tag{7-10}$$

where, V_R and V_L represent the coefficient of variation of R and L respectively.

Knowing β the probability of failure P_f can be obtained from the following equation for each limit state:

$$P_f = \Phi(-\beta) \tag{7-11}$$

The above equation is exact if the limit state function is linear and all probability distributions are jointly normal or lognormal(Ang et al. (1984), Ellingwood et al. (1980), Warner et al. (1968).

The general approach can be applied to a specific case study by evaluating the probability of site acceleration based on procedures developed for performing site seismicity studies Ferritto (1993). The site ground motion should be based on historical and geologic data for the region and reflect local site soil conditions. The ultimate capacity of the structure must be determined. Measures of uncertainty need to be established for both the load and the capacity.

Reliability Methodology For Seismic Loads

Wen et al.(1994) suggest the calculation of probability of failure based on the following equation. This is similar to the equation developed by Ang and Tang (1984)

$$P_f = \Phi\left(-\frac{\ln(SA_c / SA_r)}{\left(\beta_c^2 + \beta_r^2\right)^{1/2}} \right) \tag{7-12}$$

where, Φ ( ) is the standard cumulative normal distribution function. SA_c and SA_r are the median values of spectral acceleration of structural capacity and load of a lognormal distribution respectively. β_c and β_r are logarithmic standard deviation for structural capacity and load corresponding to a lognormal distribution.

In this case the median value of spectral acceleration is determined by means of the capacity spectrum method Freeman (1978), Wen et al. (1994). The median value of the spectral acceleration of load SA_r given by Wen et al. (1994),

$$SA_r = (SA_n) * (A_p) \qquad (7\text{-}13)$$

A_p is the value of peak ground acceleration (PGA) and SA_n is the median normalized spectral acceleration determined from response spectra in the Tri-services guidelines (5). The calibration of seismic structural design parameters as related to reliability based design is discussed in detail elsewhere, Han et al. (1994), Wen (1994).

Detailed Development Of Methodology For Seismic Loads

The following outlines steps suggested for use as a general procedure for the seismic reliability analysis of waterfront construction and used in the case study of a wharf reported in following sections.

1. The uncertainty in structural loading is obtained by first identifying the level of the earthquake. There are two levels of earthquake that are used for waterfront structures. One is the Level 1 event often termed OLE (Operating level earthquake) and the other is the level 2 event often termed CLE (Contingency level earthquake) . The first one has a probability of exceedance of 0.5 in 50 years and the second one has a probability of exceedance of 0.1 in 50 years. Using this information on acceleration, obtain the corresponding mean value of acceleration and the 95% confidence limits from cumulative acceleration plots. This uses the general procedures for computing site seismicity and seismic hazard analysis, Ferritto (1993), Sykora (1989).

2. From the mean value and 95% confidence limits of acceleration calculate the corresponding standard deviation of acceleration for a normal distribution. This will be $\overline{L}$ and σ_L.

3. Identify the limit state of the structure which controls capacity.

4. Identify the random parameters in the structure capacity. The uncertainty in structural capacity is obtained by first identifying all the random properties to be included. The geometric properties may be treated as deterministic variables, as was done in the case study in the following section. The material properties are treated as random variables. In the case study the two random variables are-- M_y (yield moment of each pile), subgrade soil stiffness(K).

5. For a set of random parameters of the variables, use an automated analysis program to compute response and collapse load. This will give a random value of the collapse load.

6. Repeat the process illustrated in step 5 for 1000 samples of random values which in turn will give 1000 random collapse loads. Monte Carlo simulation, Ang et al. (1984), Warnet et al. (1968) is used for this purpose.

7. For each random collapse load calculated in steps 5-6 calculate the corresponding random value of capacity acceleration.

8. Calculate the mean and standard deviation of all the random values of accelerations. This gives $\overline{R}$ and σ_R.

9. From the results of steps 2 and 8 calculate the safety index β, for the collapse limit state considered, from Equation 7-9 for normal distribution.

10. The probability of failure is also obtained from Equation 7-12 or Equation 7-10 if the distributions of accelerations for capacity and loading are assumed as lognormal.

Wharf Reliability Demonstration

The Navy has recently completed design for dredging and construction of a carrier wharf at the Naval Air Station, North Island, California. This project is typical of wharf design and was used in a simple form as a demonstration study to illustrate the procedures discussed above. Since the wharf model incorporated a number of simplifications and assumptions where actual soil data was not available, it should not be looked upon as a performance evaluation of the actual construction project.

Regional Seismicity Required

A reliability analysis requires quantification of the seismic load environment and its associated uncertainty. To that end a seismicity study must be performed. The results of such a study are discussed in this section.

The seismicity and regional geologic structure of the San Diego area can be interpreted in light of current plate tectonic theory. California lies on the junction of two relatively rigid plates of the earth's crust that respond to movement of subcrustal material. The main evidence of this juncture is the San Andreas fault. These same forces that tend to move the portion of California on the westerly side of the San Andreas fault northward have resulted in the formation of other faults, such as the San Jacinto, Whittier-Elsinore and Newport-Inglewood faults. Distant faults that must be considered significant to the site region include the Elsinore and San Jacinto fault zones to the northeast and the San Clemente fault zone to the west. Local faults include the Rose Canyon and La Nacion. The San Andreas fault zone is not considered very significant because of its great distance from the study area. This section is based on a detailed study performed by Ferritto (1994).

The San Diego Bay contains Cretaceous, tertiary, and quaternary strata, which is generally flat but locally folded and cut by normal and right lateral faults. This area is called the Rose Canyon zone Lee et al. (1988). A bottom survey of the bay revealed numerous faults which were difficult to correlate. The quaternary deformations observed along the Rose Canyon fault zone attest to the tectonic importance of the zone. Although no major earthquakes have occurred near San Diego recently, several earthquakes of about magnitude 3.5 have been recorded during the past 41 years. Eleven took place near the Rose Canyon fault. The magnitude 3.5 earthquake is associated with a fault rupture length of 1 km. The geologic structure of this area shows evidence of previous movement. Surface traces of more than 24 km in length and vertical separation of hundreds of feet are visible. Table 1 shows the key faults and the maximum credible earthquake.

Probability Analysis

The bounds of the study area are 115.0 to 119.0 W longitude, 34.0 to 32.0 N latitude. The coordinates of the site are 117.18N, 32.705N. A set of historical data was prepared for the site containing over 6,000 events with magnitudes of 3 or greater. Figure 7-1 shows the region of interest with the epicenters plotted. Figure 7-2 shows a similar plot with only the faults shown. Figure 7-3 shows the total probability of not exceeding the acceleration for a 50-year exposure.

The best estimate of site seismic exposure from all sources is as follows:

1000 year	0.60 g
500 year	0.42 g
250 year	0.28 g
100 year	0.18 g

For the purpose of engineering analysis the causative events are as follows:

- The 1000 year earthquake is a magnitude 6.5 event at 1 to 3 miles from the site.
- The 500 year earthquake is a magnitude 5.5 event at 1 to 3 miles from the site or a magnitude 6. to 7 event at about 10 to 20 miles from the site.
- The 250 year event is a magnitude 5 event at about 2 miles from the site.

The seismicity at the site is totally dominated by the Rose Canyon fault. Generally the causative events associated with ground motion return times specified are caused by magnitude 5 to 5.5 earthquakes close to the site. These events would not have durations as long as those associated with magnitude 6 to 7 events. As noted there is the possibility of magnitude 6 to 7 events 10 or more miles from the site which would produce longer duration shaking. To support this study a

Table 7-1
Fault Systems of Interest

Fault	Maximum Credible Magnitude
Coyote Creek	7.0
Elsinore	7.5
Imperial	7.0
La Nacion	6.8
Malibu	7.5
Newport-Inglewood	7.0
Palos Verdes	7.0
Pinto Mountain	7.5
Raymond Hills	7.5
Rose Canyon	7.1
San Clemente	7.7
San Gabriel	7.7
San Jacinto	7.5
Santa Susana	6.5
Sierra Madre	6.5
South San Andreas	7.5
Superstition Mountain	7.0

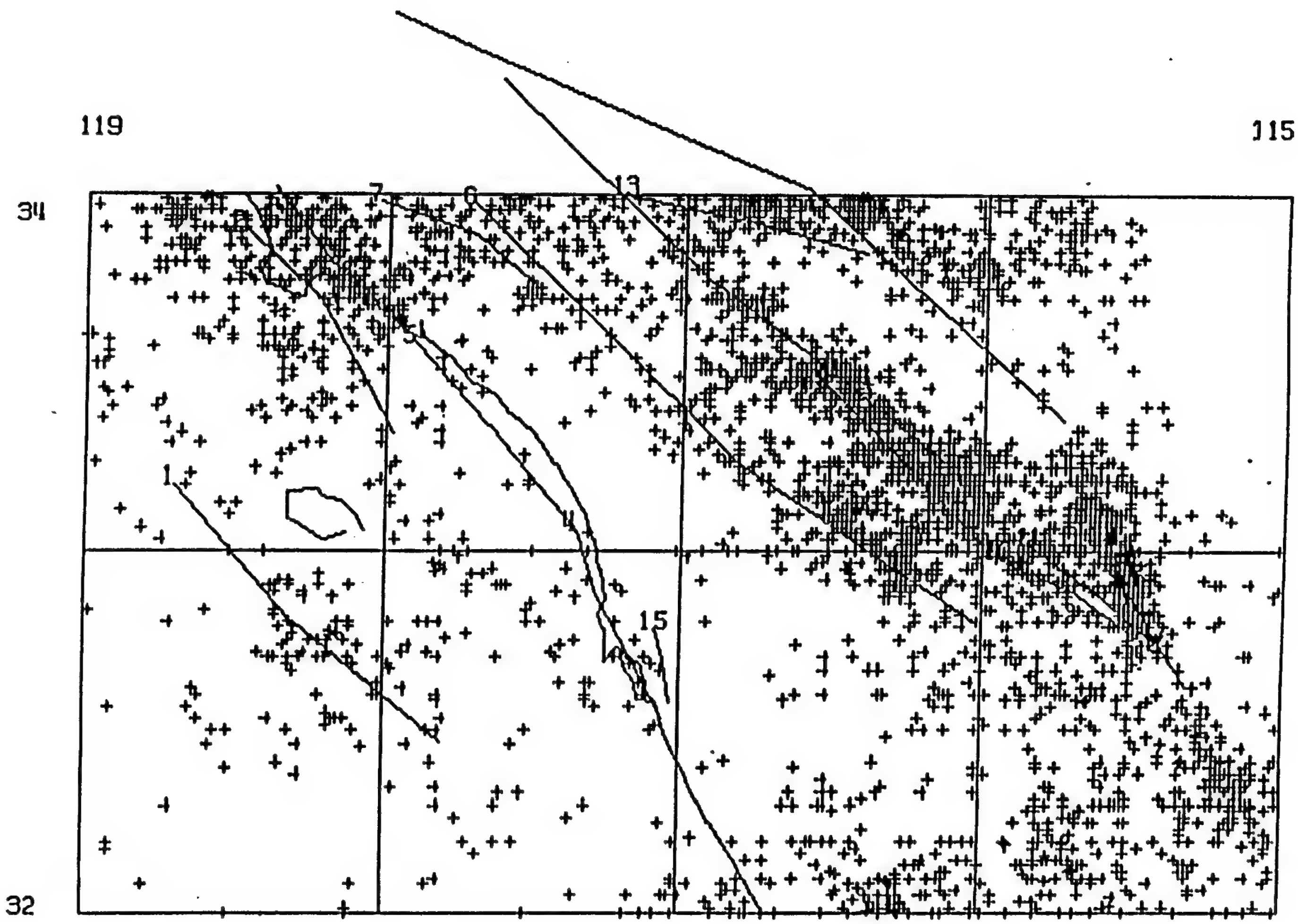

Figure 7- 1. Region surrounding site showing faults and historical epicenters.

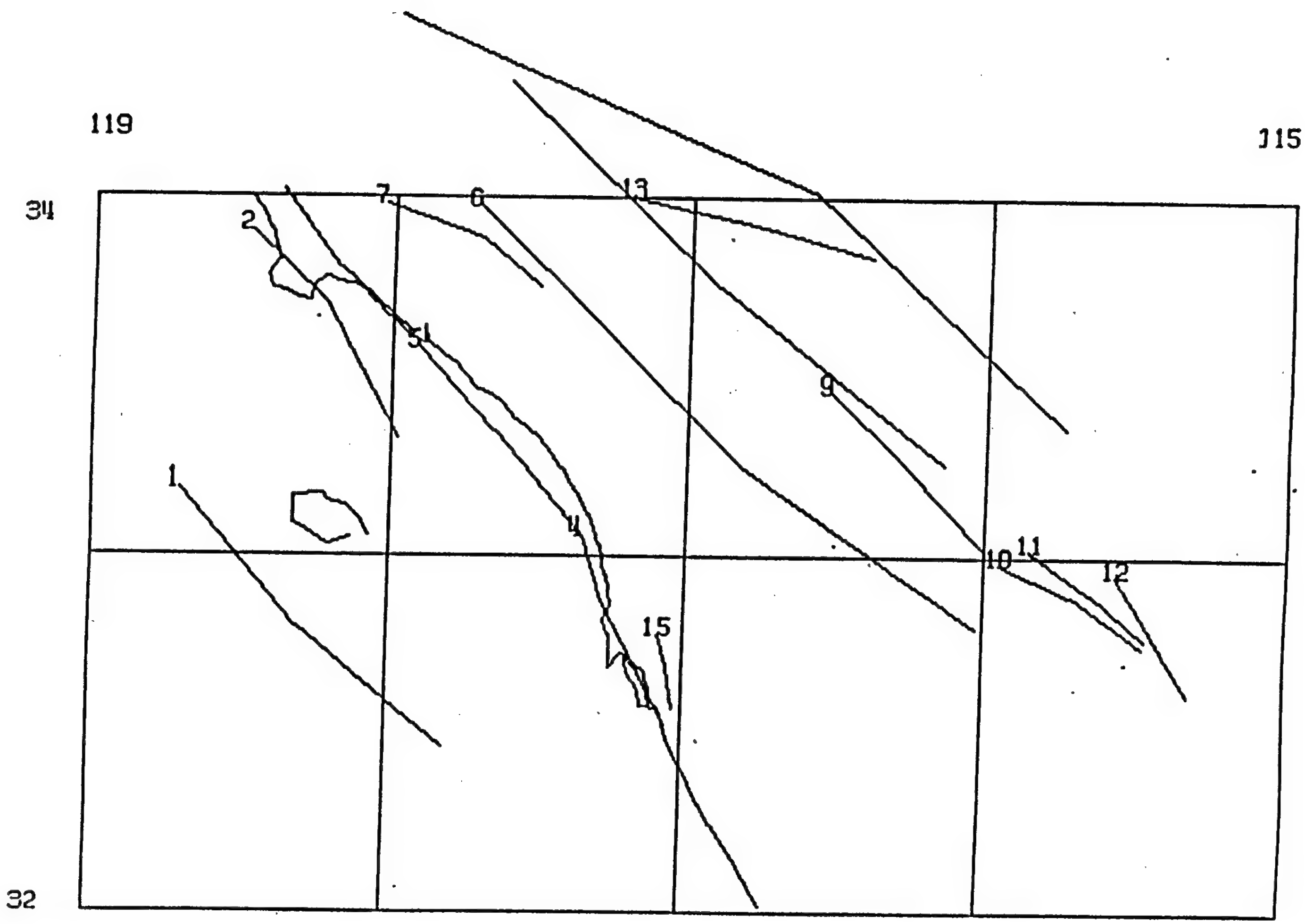

Figure 7-2. Region surrounding site showing faults as defined in Table 1.

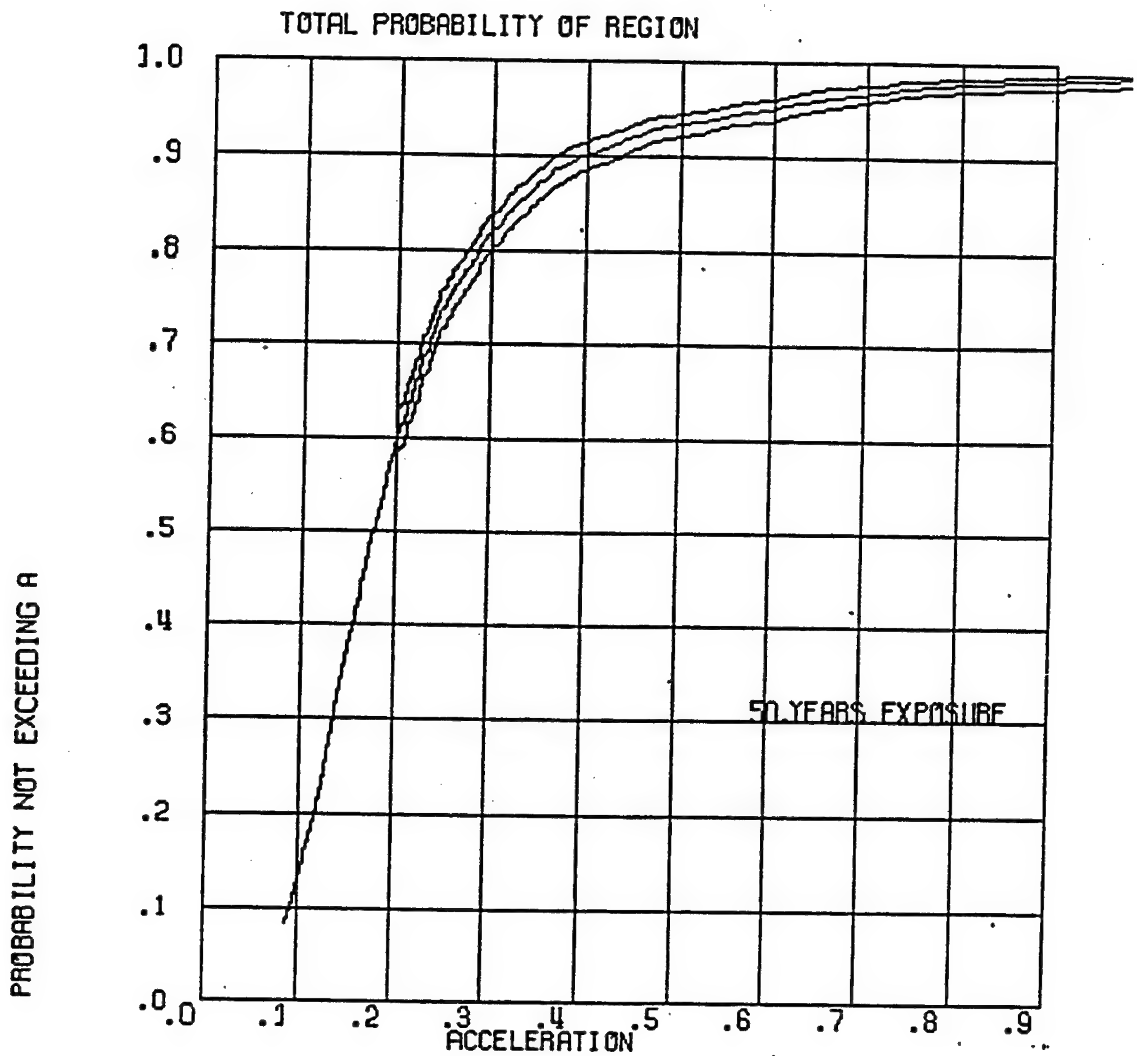

Figure 7-3. Probability distribution of site peak acceleration.

number of earthquake time history records were selected using procedures documented in Ferritto (1992).

Site Soil Model For Ground Motion Amplification/Attenuation

The following site soil foundation model was developed for this study to model a section of the dike below the wharf and soil layers below.

Profile	Thickness ft	Density Lb/cu ft	Blow Count	Shear Modulus fps
Rock Dike	50	140		1500
Bay Point Formation Layer 1	10	120	40	1040
Bay Point Formation Layer 2	10	120	60	1200
Bay Point Formation Layer 3	600	120	80	1400-3000

The blow count data was used to establish the shear velocity and shear modulus using data from Sykora (1989). The shear modulus was allowed to increase with depth to bedrock. It was also decided to use mean values for the shear modulus and damping relationships as a function of strain rather than lower bound values. A one dimensional wave propagation analysis was performed to estimate the acceleration time history in the rock dike using the established level of seismicity as a bedrock acceleration. A series of records were used to represent possible ground motion variation and the variance determined. **This example is based on an existing project which used the 1,000 year event as the design earthquake rather than the 500 year event suggested for use in the criteria section.** It is recommended that the events shown in the criteria be used and the data used herein is intended only to demonstrate the methodology. The 1,000 year peak acceleration earthquake level motion using the 1-dimensional wave propagation analysis was computed for each of the records; the average acceleration is 0.5g with a standard deviation of 0.14 g. The motion is seen to be transmitted to the surface with some attenuation from the rock motion of 0.6g. The uncertainty of the level of this motion was computed. This uncertainty was combined with the uncertainty of value from the seismicity study The values to be used for the reliability analysis are:

1,000 Year Peak Acceleration Mean Value 0.5g

1 σ Standard Deviation 0.147g

Example Wharf and Lateral Resistance Structural Model

The example wharf is shown in Figure 7-4. It was decided to model the structure in two dimensions using the typical cross-section shown in Figure 7-4. The structure is composed of a reinforced concrete deck supported on pile caps. The first pile on the land side is a 28-inch diameter steel pipe pile filled with concrete. The next four piles are 24-inch octagonal prestressed concrete piles and the outboard fender pile is a square 24-inch prestressed concrete pile. A preliminary analysis showed that 90 percent of the lateral resistance was provided by the 28-inch

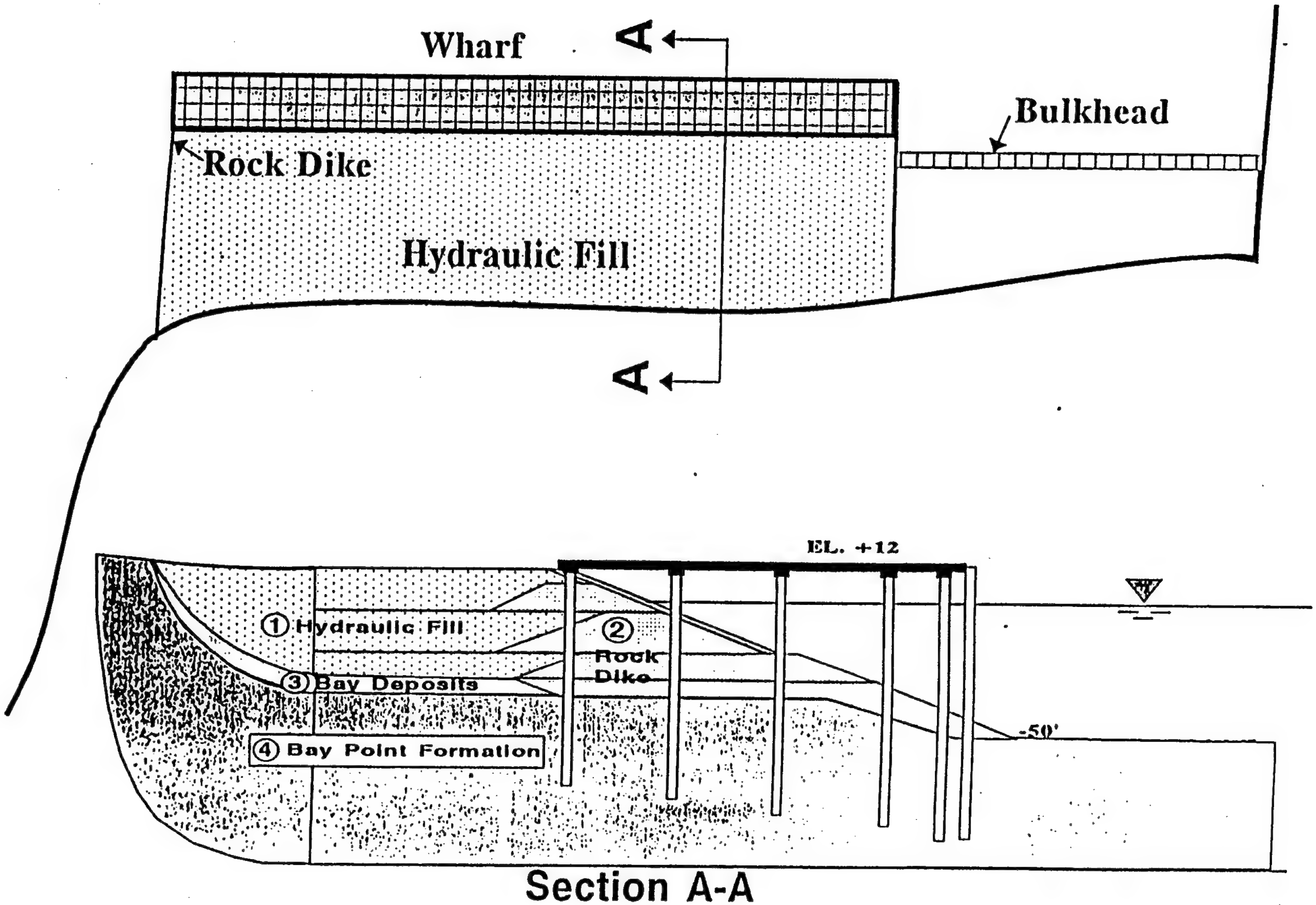

Figure 7-4. Plan view of wharf and dike used in study.

diameter steel pipe pile filled with concrete. The lateral force model of the wharf could thus be simplified to model the pile as a series of beams, constrained by the deck and supported by lateral springs representing the dike. The structural model and procedure used for this study, utilized the ultimate moment - thrust capacity of the pipe pile to define the lateral force capacity. The mass of the structure including deck, piles and restraining soil around the piles was computed. A reliability analysis requires computation of the mean value of capacity and an estimate of its uncertainty. The variance of pile capacity was estimated to be 0.15 and the variance in soil stiffness subgrade modulus for lateral pile restraint was estimated to be 0.10, Arbabi et al. (1991), Ellingwood et al. (1980). The weights of the wharf itself could be estimated with a high degree of reliability. An amount of soil representing the lateral spring stiffness of the dike was included; this could only be determined approximately. The uncertainty of this soil mass was set by giving it a variance of 0.5.

Results Of Reliability Analysis And Discussion

For the analysis conducted the following was found:

OLE Operating Level Event
100 year return time ground motion

Loads, 0.18g $\sigma = 0.07g$
Capacity 1.248g $\sigma = 0.382g$

$\beta = 2.72$ $P_f = 0.003$
For normal distribution

$\beta = 4.09$ $P_f = 0.00002$
For lognormal distribution

CLE Contingency Level Event
1000 year return time ground motion

Loads, 0.5g $\sigma = 0.147g$
Capacity 1.248g $\sigma = 0.382g$

$\beta = 1.827$ $P_f = 0.034$
For normal distribution

$\beta = 2.20$ $P_f = 0.014$
For lognormal distribution

The reason for choosing the normal distribution for either structural capacity or loads is mainly based on the principle of maximum entropy , Harr (1987). Based on this principle the normal distribution is to be assumed if the expected value and standard deviation of a distribution are the known parameters. Further it has been stated by Ang and Tang (1984) that the normal or lognormal distribution is frequently used to model non-deterministic problems even when there is no clear basis for such a model. Since almost all of the random data for structural capacity and load are positive it was decided to use a lognormal distribution in addition to the normal distribution. The decision to use a lognormal distribution in addition to a normal distribution is also based on the recent research by Wen et al. (1994) wherein they advocate use of the lognormal distribution for structural capacity and loads in connection with seismic studies. The lognormal distribution computes lower probabilities of failure and is thought to be a more accurate estimate of the results for the seismic study which is consistent with current practice Turkstra et al. (1978), Wen et al. (1994).

The results indicate that the probability of failure under the operating load to be about 0.003 or 0.3 percent for the normal distribution and 0.00002 or 0.002 percent for the lognormal distribution. The probability of failure under the collapse level of loading is about 0.034 or 3.4 percent for normal distribution and 0.014 or 1.4 percent for lognormal distribution. The uncertainty in both loading and capacity was found to be significant as can be seen by the high coefficient of variation values. The uncertainty levels for structural capacity and loading computed in this report are in the same range as in recent report by Wen et al. (1994). A major element in the uncertainty is the manner of wharf -dike coupling. The procedure developed requires the computation of the mean collapse level capacity and its uncertainty. This can not be computed directly in a closed form manner and use of a finite element program is required. This project is of limited scope, intended to demonstrate the feasibility of a general procedure; the analysis options were constrained by available project duration and funding. Determination of the collapse load can be performed by an equivalent static lateral load model as was done here or a more elaborate dynamic soil structure interaction model. The Monte Carlo procedure requires repetition of the analysis varying the strength parameters to evaluate the mean and variance of the capacity. Typically repetitions on the order of 1,000 are used. This poses a problem for implementation of a dynamic finite element approach. The equivalent static approach is thought most appropriate. A major factor in the analysis as shown by the sensitivity of results to the variance in capacity is the estimate of the mass of the system to be used to compute the equivalent capacity acceleration. The effective mass of the soil coupled to the pile was estimated to be a region associated with the pile about 1.5 pile diameters wide by about 3 pile diameters long for the length of the pile. A 50 percent uncertainty was assigned to this soil weight to account for this uncertainty. It should be noted that the more soil mass that is included the lower is the equivalent lateral force capacity. This aspect should be given additional study using a dynamic soil-structure model of the problem to verify the mass effect.

The results show that for the two load conditions specified the probability of collapse is between 0.00002 and 0.003 under the operating level and between 0.014 and 0.034 percent under the contingency level. The wharf design allowed possible major repairable damage under the CLE. This case study is meant only as an illustrative example and is loosely based on the design of the wharf at Naval Air Station, North Island. Thus direct conclusions about the actual wharf

should not be made. From this simplified model, it would appear that the wharf in this case study would be expected to perform well under both the OLE and the CLE event.

The safety index β values, for the capacity and load having normal and lognormal distributions, have been calculated using uncertainties in these parameters for a typical wharf structure subjected to seismic loads. Both the OLE (Operating Level Event) and the CLE (Contingency Level Event) are considered in this study. The uncertainties in capacity and load parameters reported in this study are consistent with other work dealing with seismic loads by Wen et al. (1994). A high value of safety index β is found to be for the OLE while a low level of safety index β is found to be for CLE. This is consistent with the fact that for CLE the safety index β should be low as it is a contingency level event.

Limitation in Analysis and Need for Additional Study

Limitations in the scope of this effort necessitated use of a simplified wharf model. A major element of uncertainty is the wharf dike response. It is possible that dike slope deformations can induce additional curvature into the piles. This aspect of soil structure interaction could not be addressed in the model used in this study. It is expected that this would be of concern only for the CLE. As suggested above a more detailed study could better evaluate the effect of dike deformation on wharf capacity.

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